Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

PRICE 2/6 MARCH 1959

CONTROL GATES

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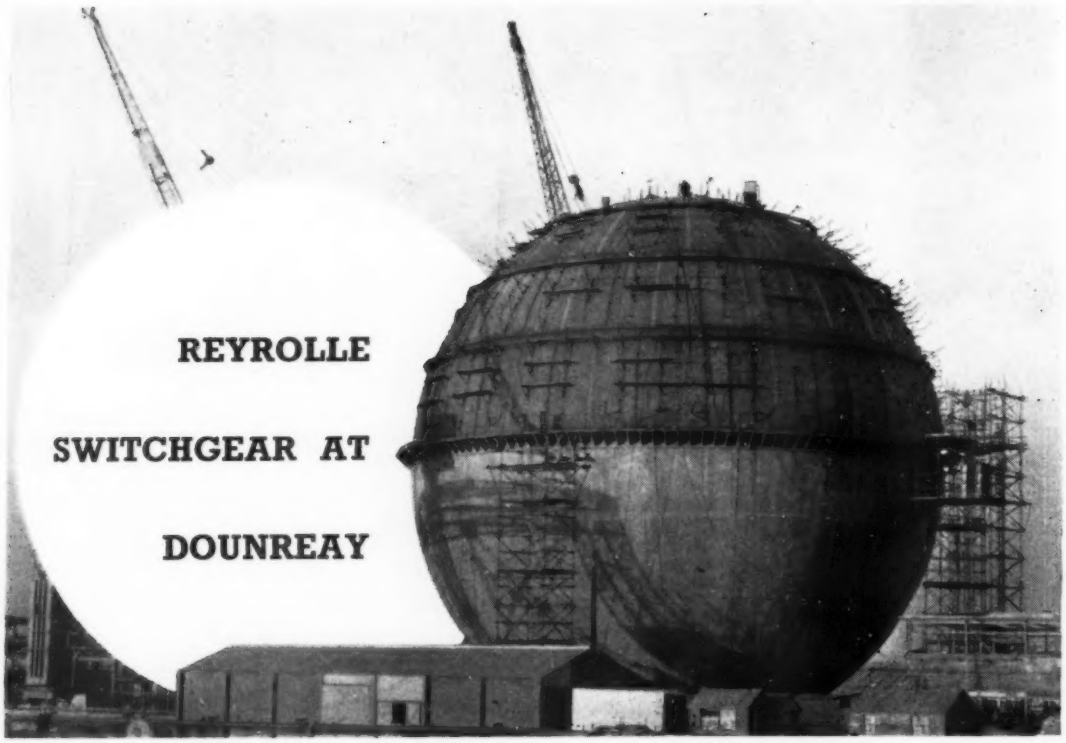
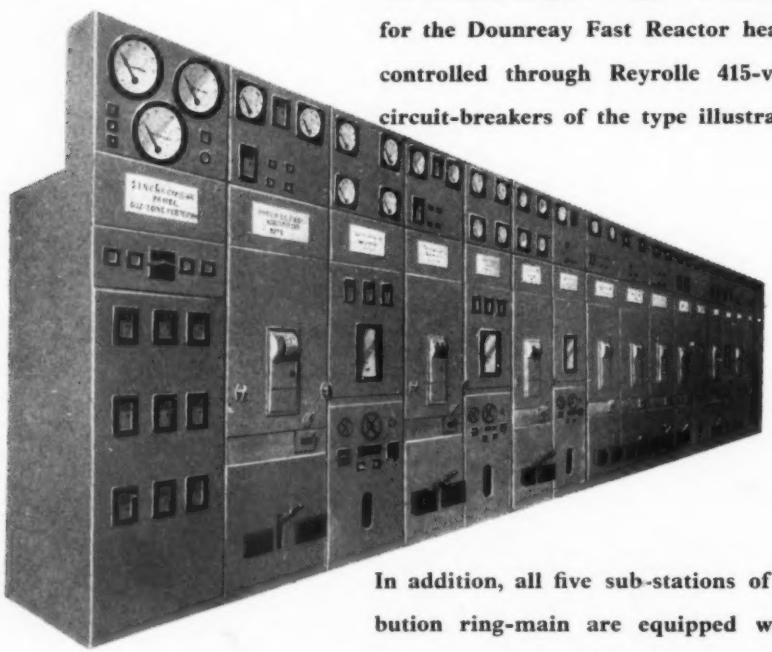
Glenfield
GLENFIELD & KENNEDY, LIMITED, KILMARNOCK



REYROLLE SWITCHGEAR AT DOUNREAY

Photograph by courtesy of the U.K.A.E.A

The electro-magnetic liquid-metal pumps and other services for the Dounreay Fast Reactor heat-exchanger system are controlled through Reyrolle 415-volt 31·5-MVA air-break circuit-breakers of the type illustrated.

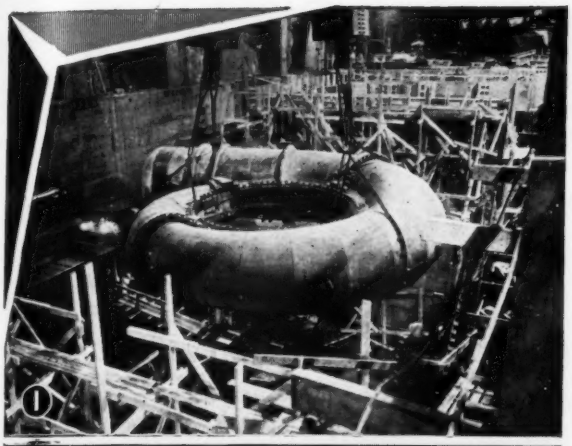


17-panel switchboard comprising Reyrolle type-S22A air-break circuit-breakers with associated control cubicles.

In addition, all five sub-stations of the main 11-kV distribution ring-main are equipped with Reyrolle metalclad switchboards with 350-MVA horizontal draw-out oil-break circuit-breakers.

Reyrolle

A. Reyrolle & Co. Ltd · Hebburn · County Durham · England



turbines for three hydro-electric schemes add over 2,000,000 h.p. to Canada's power potential

For three of the world's largest hydro-electric projects 26 water turbines of 'ENGLISH ELECTRIC' design are being supplied by English Electric Canada, a division of John Inglis Co. Ltd. and manufactured at Toronto. These machines will add to Canada's power potential a total of over 2,000,000 h.p. Of this over 1,000,000 h.p. is already in operation.

1 **BERSIMIS**—WORLD'S LARGEST HIGH HEAD TURBINES DEVELOP RECORD OUTPUT

Throughout the past year at Bersimis-Lac Casse Development 520,000 kW has regularly been supplied to Hydro Quebec's power system from four 'ENGLISH ELECTRIC' water turbines. On test these turbines have developed 200,000 h.p. under a net head of 840 feet, thus establishing a world record for hydro-electric units.

2 **NIAGARA**—OUTSTANDING SUCCESS OF WORLD'S FIRST DERIAZ REVERSIBLE PUMP/TURBINES

Six 'ENGLISH ELECTRIC' Deriaz reversible variable pitch pump/turbines are in commercial operation at the Sir Adam Beck-Niagara Pumping Generating Station operated by the Hydro-Electric Power Commission of Ontario. Operating under 83 feet head, each machine is rated at 45,500 h.p. as a turbine and as a pump will discharge 4,000 to 5,000 cubic feet of water per second under a variable head. These pump/turbines are of a new basic design developed by 'ENGLISH ELECTRIC' and have successfully absorbed 65,000 h.p. when pumping quantities in excess of 6,000 c.f.s. against a dynamic head of 80 feet.

3 **ST. LAWRENCE POWER PROJECT**—SIXTEEN 75,000 h.p. TURBINES

For Ontario Hydro's Power Project on the St. Lawrence, sixteen 'ENGLISH ELECTRIC' 75,000 h.p. water turbines were ordered in 1955. The first four machines went into commercial operation in July, 1958. The power generated will be Canada's share of the output from this international power project in conjunction with the St. Lawrence Seaway.

'ENGLISH ELECTRIC'
water turbines

THE ENGLISH ELECTRIC COMPANY LIMITED, MARCONI HOUSE, STRAND, LONDON, W.C.2

ENGLISH ELECTRIC CANADA (A DIVISION OF JOHN INGLIS CO. LTD.) ST. CATHARINES, ONTARIO

Offices and representatives throughout the world

HE. 32 A

★ The illustration shows a Horseley - Piggott Pressed Steel Tank which was supplied to the Dearne Valley Water Board.

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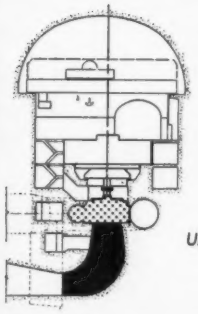

SECTION THROUGH UNDERGROUND POWER HOUSE

The first stay ring for the Boving turbine of No. 1 unit at the Kariba underground power station, was shipped in May 1958. It is shown here, prior to shipment, at the works of Markham & Co. Ltd., Chesterfield, complete with guide apparatus assembled.

Output 140,000 H.P.
Normal speed 167 r.p.m.
Max. nett head 340 ft.

Boving
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WATER POWER ENGINEERS

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The Voith Hydraulic Research Laboratories provide ideal test facilities for turbines, pumps and hydraulic steel structure equipment

Aerial view of the "Brunnenmühle" Hydraulic Research Laboratory with high-level reservoir, visible in the upper right-hand corner.
(Bertram-Luftbild, München-Riem, No. 6/4)

"Brunnenmühle" Hydraulic Research Laboratory:

About 7 cusecs of clear spring water under 6 ft. head or 35,000 cusecs of water from a high-level reservoir under 328 ft. head are available for investigating all types of turbines, shut-off valves, weirs and pumps. A separate cavitation test stand, allowing variation of the suction head at constant total head, and aerodynamic test stands complete our most modern testing facilities.

"Hermaringen" Hydraulic Research Laboratory:

160 cusecs are available under 19 ft. head for testing low-head turbine models of large dimensions. In both laboratories numerous test stands are provided for varied testing and research work. For full particulars, see our publication "Voith Forschung und Konstruktion," Vol. 4.

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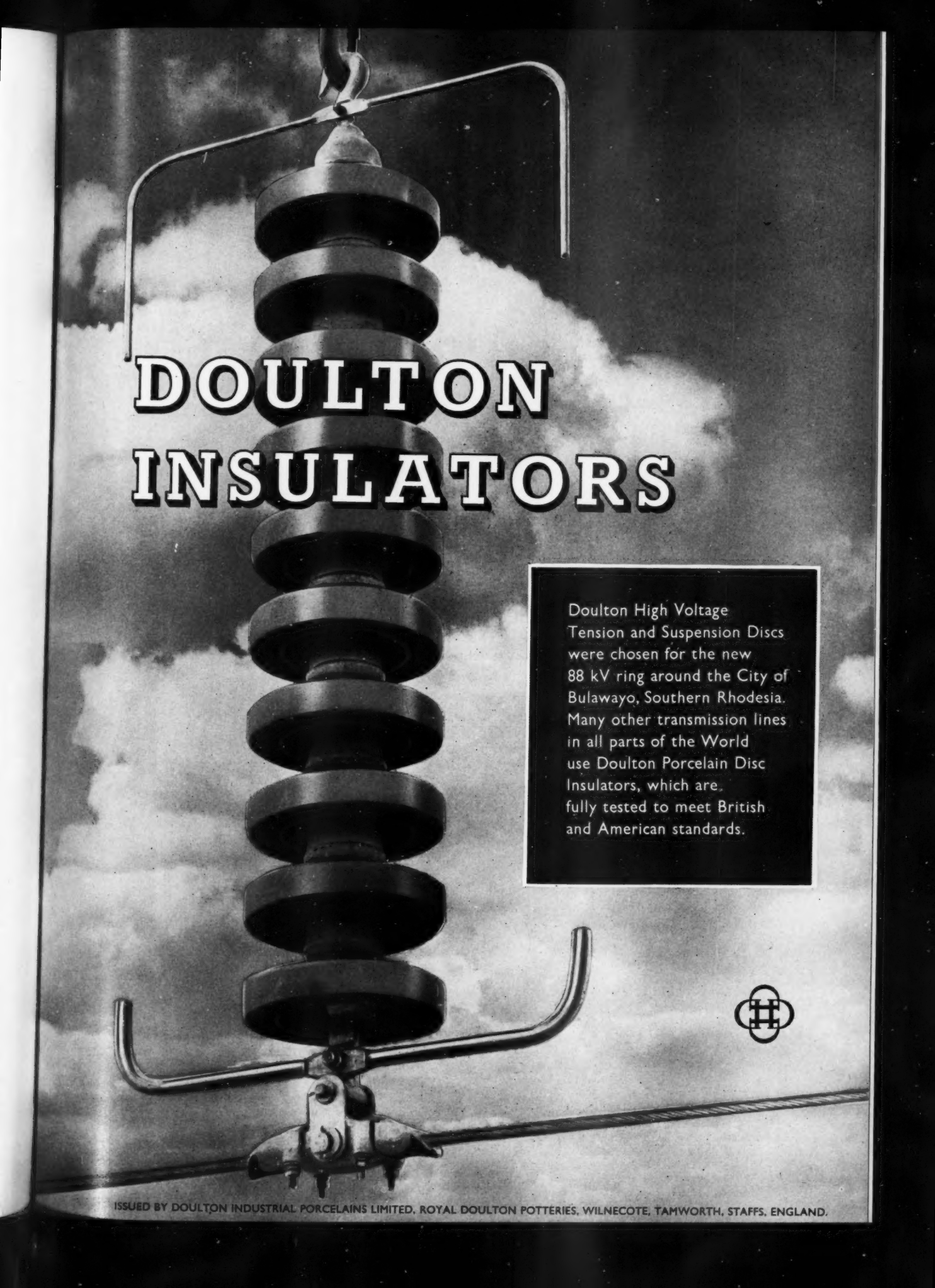
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Doulton High Voltage Tension and Suspension Discs were chosen for the new 88 kV ring around the City of Bulawayo, Southern Rhodesia. Many other transmission lines in all parts of the World use Doulton Porcelain Disc Insulators, which are fully tested to meet British and American standards.
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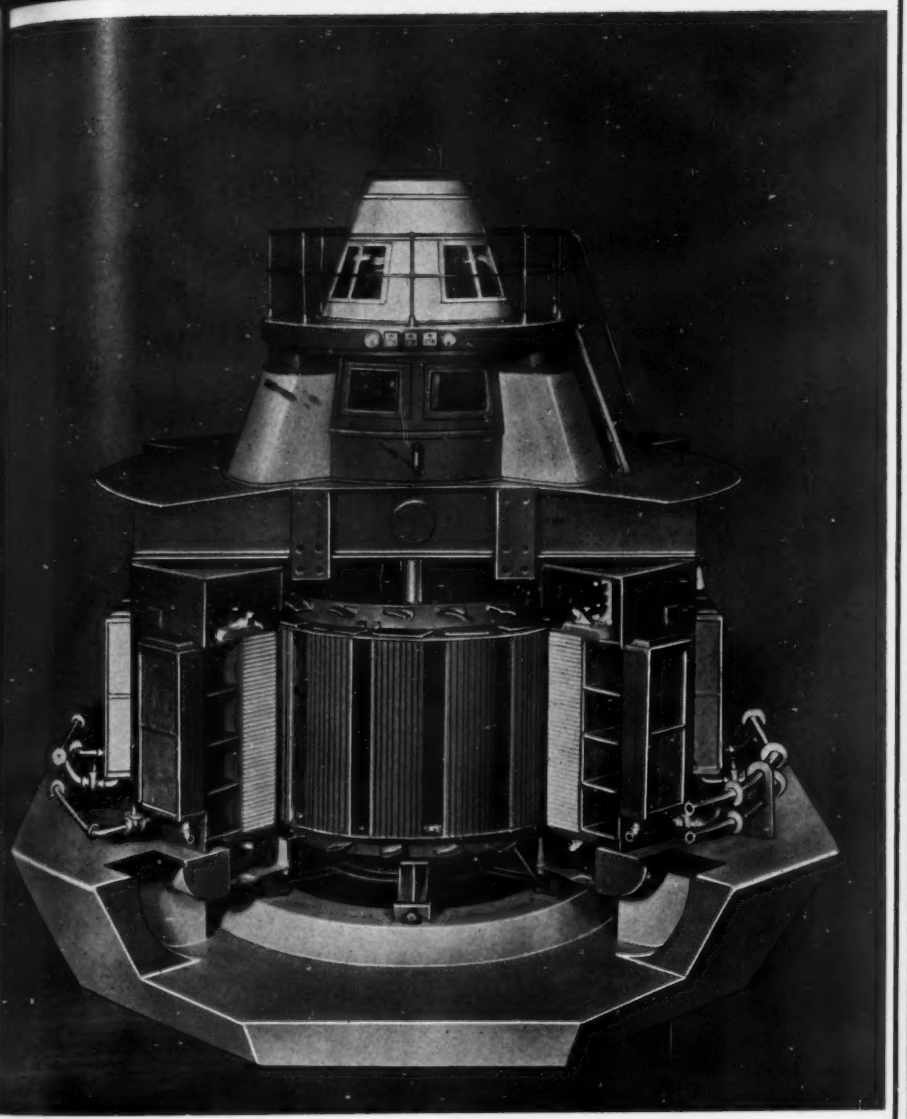

4 × 84,210 + 4 × 75,700 kVA for Australia

In the face of very keen international competition ASEA was awarded in 1954 by the Snowy Mountains Hydro-Electric Authority, Sydney, Australia, the contract for four three-phase generators, each with a rating of 84,210 kVA, 375 r/m, for the T 1 power-station on the River Tumut. In 1957 this contract was followed by yet another for four generators, each rated at 75,700 kVA, 428 r/m, for the T 2 power-station on the same river. These power-stations form part of the largest civil-engineering project ever to have been carried out in Australia, and the machines are indeed the largest water-turbine-driven generators in the Southern Hemisphere.

ASEA has now delivered or is in the process of manufacturing thirty generators with ratings exceeding 75,000 kVA. The average output of these machines is almost 94,000 kVA, and the three largest, which each have an output of 150,000 kVA, are believed to be the biggest in the world at present.

If you are considering the expansion of your present plants or the construction of new power-stations, it is in your interest to consult us. We have the experience, resources and delivery capacity to satisfy your requirements.

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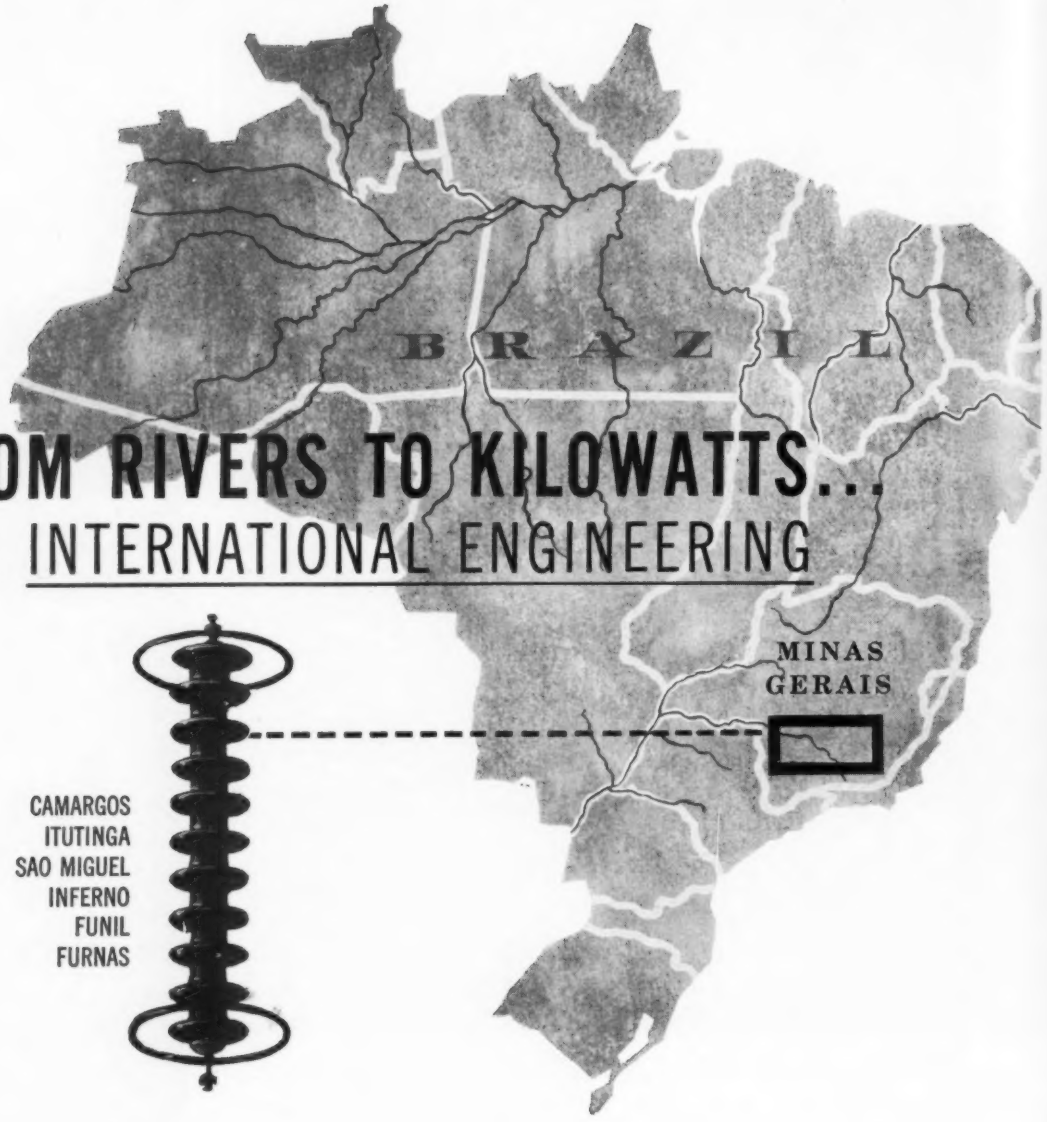
Västerås ASEA Sweden



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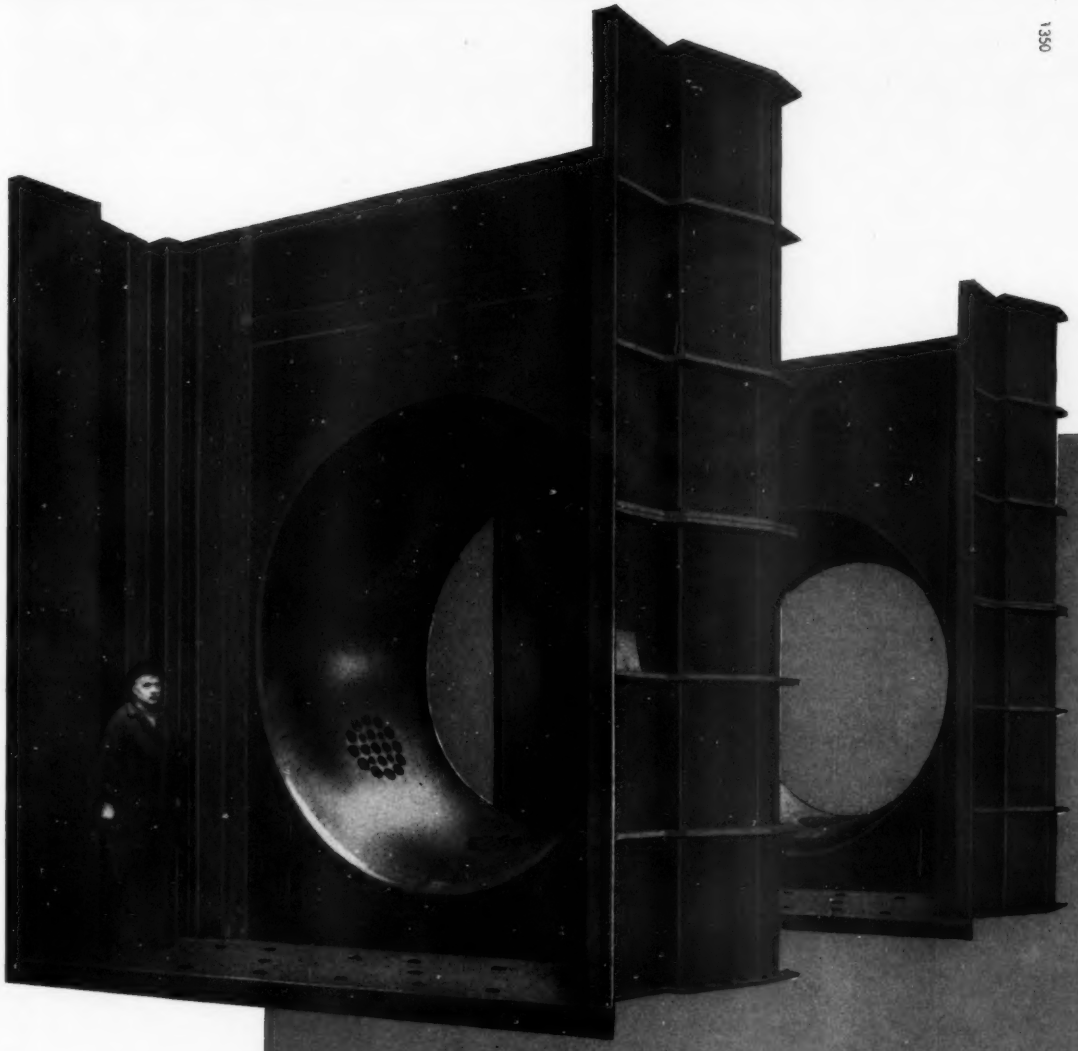

Entrance to the steel lined portion of irrigation supply tunnels for Dokan Dam, Iraq, 3,05-metre diameter bellmouth with channels to receive emergency gate, designed to operate under unbalanced water pressure of approx. 95 metres.

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of FERRANTI TRANSFORMERS
for the NIAGARA POWER PROJECT
LEWISTON, U.S.A.
FERRANTI
Ferranti Ltd. have been awarded a contract by the Power Authority of the State of New York to supply thirteen generator transformers for the Lewiston Power Station of the Niagara Power Project situated on the plateau on the Niagara River about 3 miles below the Falls in the town of Lewiston.
The contract covers six 180,000 kVA, 13.2/240 kV 3-phase transformers and seven 180,000 kVA, 13.2/120 kV 3-phase transformers. They will be of the forced-oil, forced-water cooled, inert gas-filled type and each will be directly coupled to a 167,000 kVA 3-phase generator.
Believed to be the largest transformer contract so far placed in Britain by the U.S.A., delivery will commence in the middle of 1959 and will be completed by the end of 1960.
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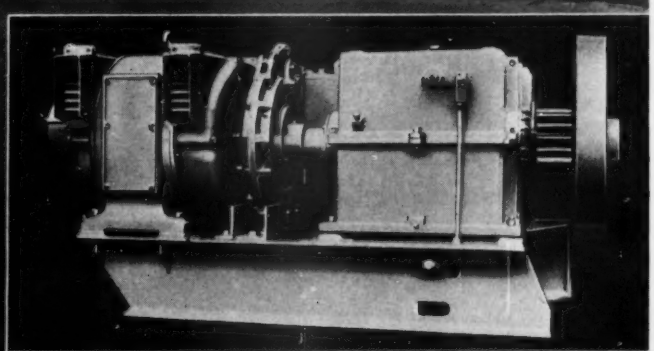

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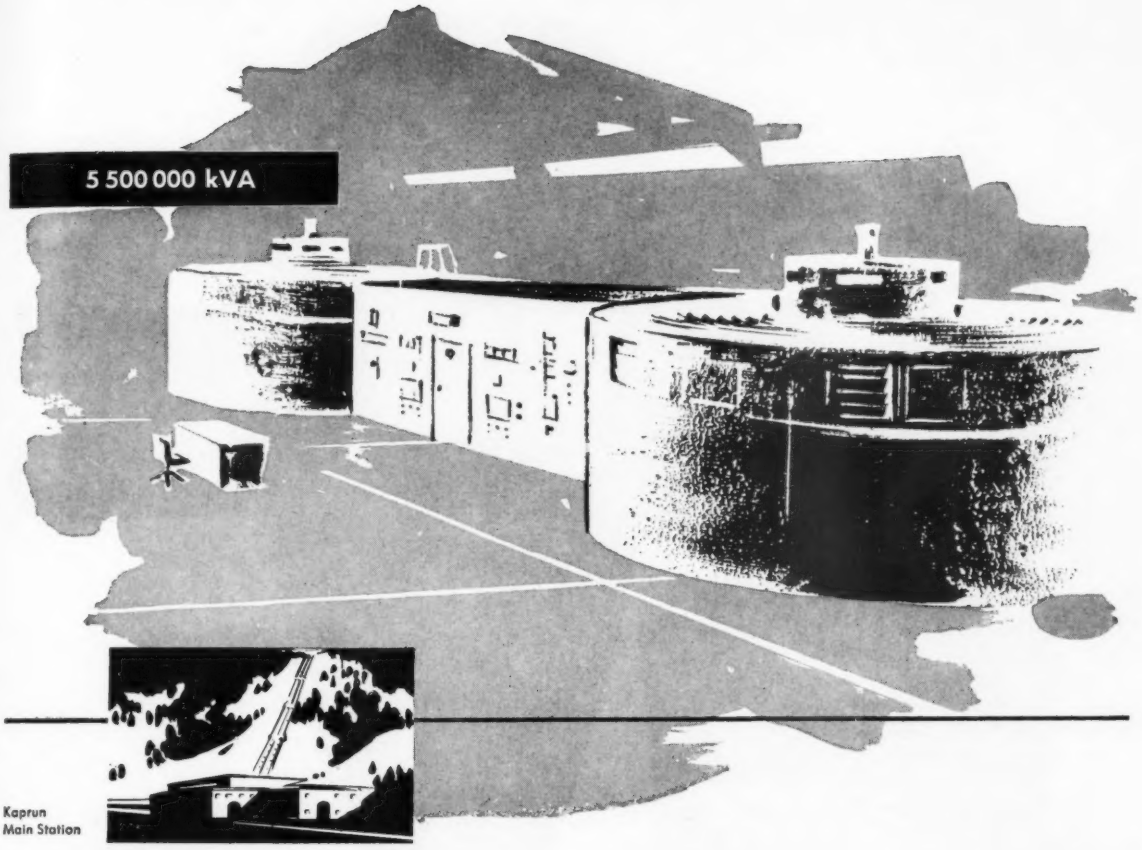
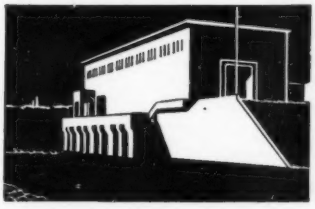
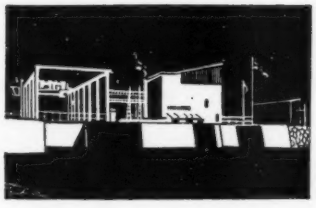


Kaprun
Main Station



Sariyar
Hydro-Electric
Power Station



Pamilo
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5 500 000 kVA
is the aggregate rating of water wheel generators AEG so far supplied or for which contracts have been placed.

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AEG

Hydro-Electric Stations

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Berlin (West) EXPORT DEPARTMENT Frankfurt (Main)

Orders received during 1958

Plant	No. & type of turbines	D nom mm	H norm. m	HP totally	n
Valajaskoski	2 Kaplan	6.500	11.2	67.800	75
Aittokoski	1 Kaplan	4.450	25.8	46.900	143
Juukoski	2 Kaplan	5.700	24.5	148.000	115

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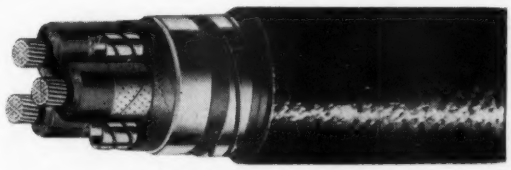

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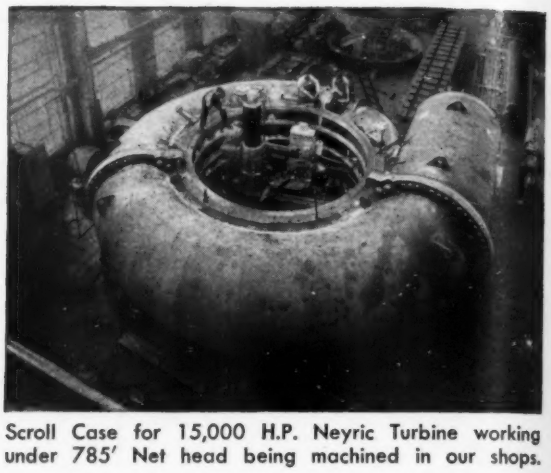
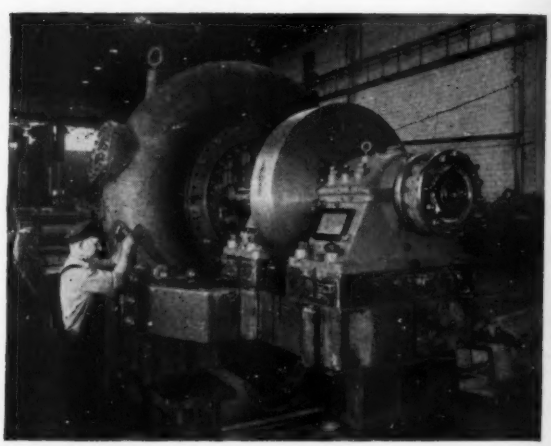
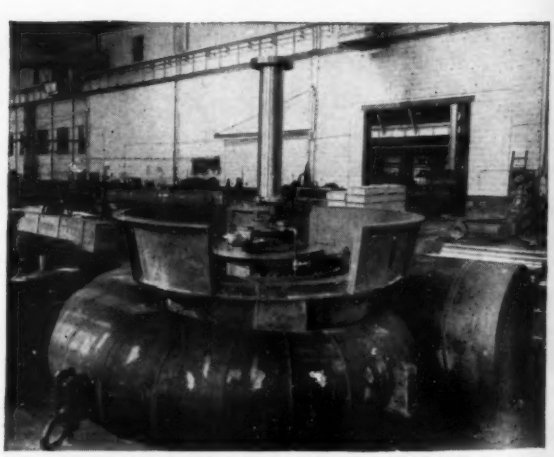


Scroll Case for 15,000 H.P. Neyric Turbine working under 785′ Net head being machined in our shops.



Shop assembly of Vertical Leffel Turbine rated 3,000 H.P. under 125′ Net head.



A Horizontal Shaft Leffel Turbine rated 3,200 H.P. under 425′ Net head, partly assembled.

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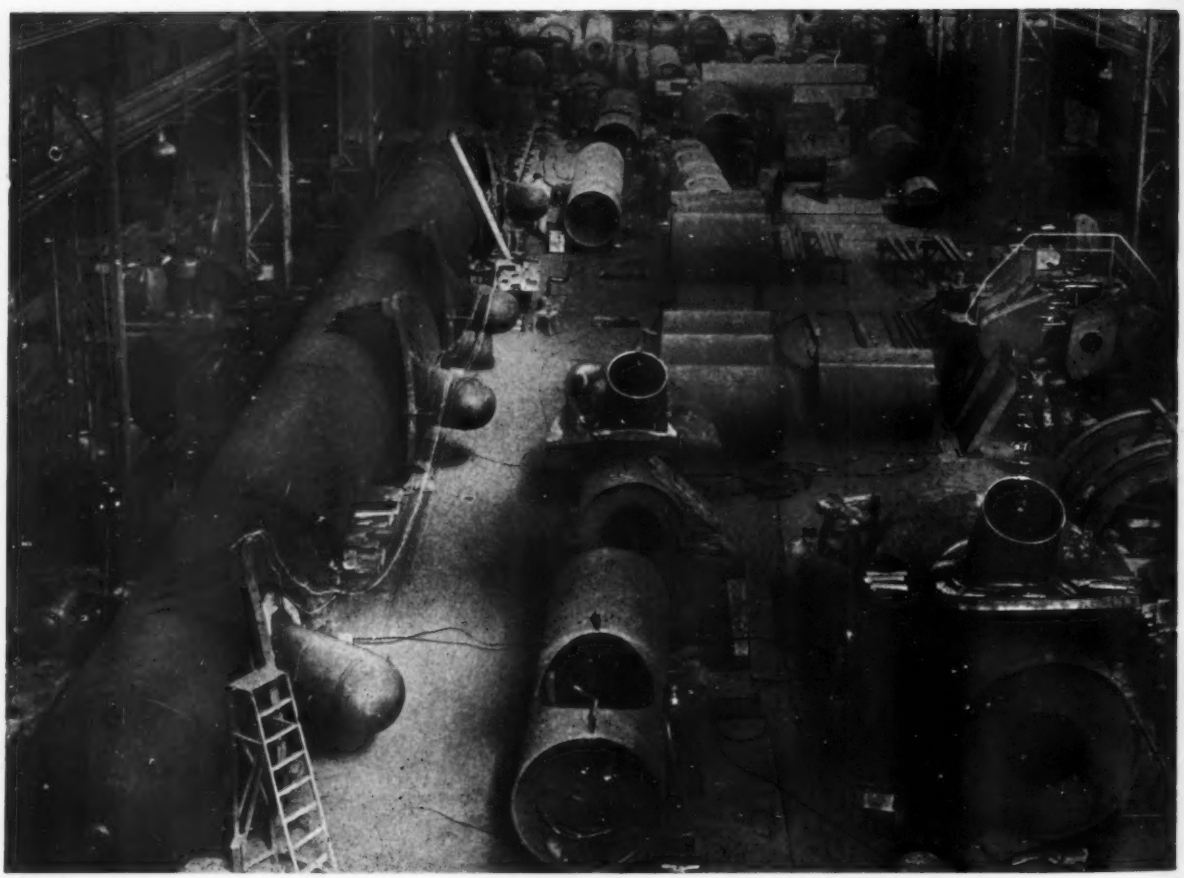
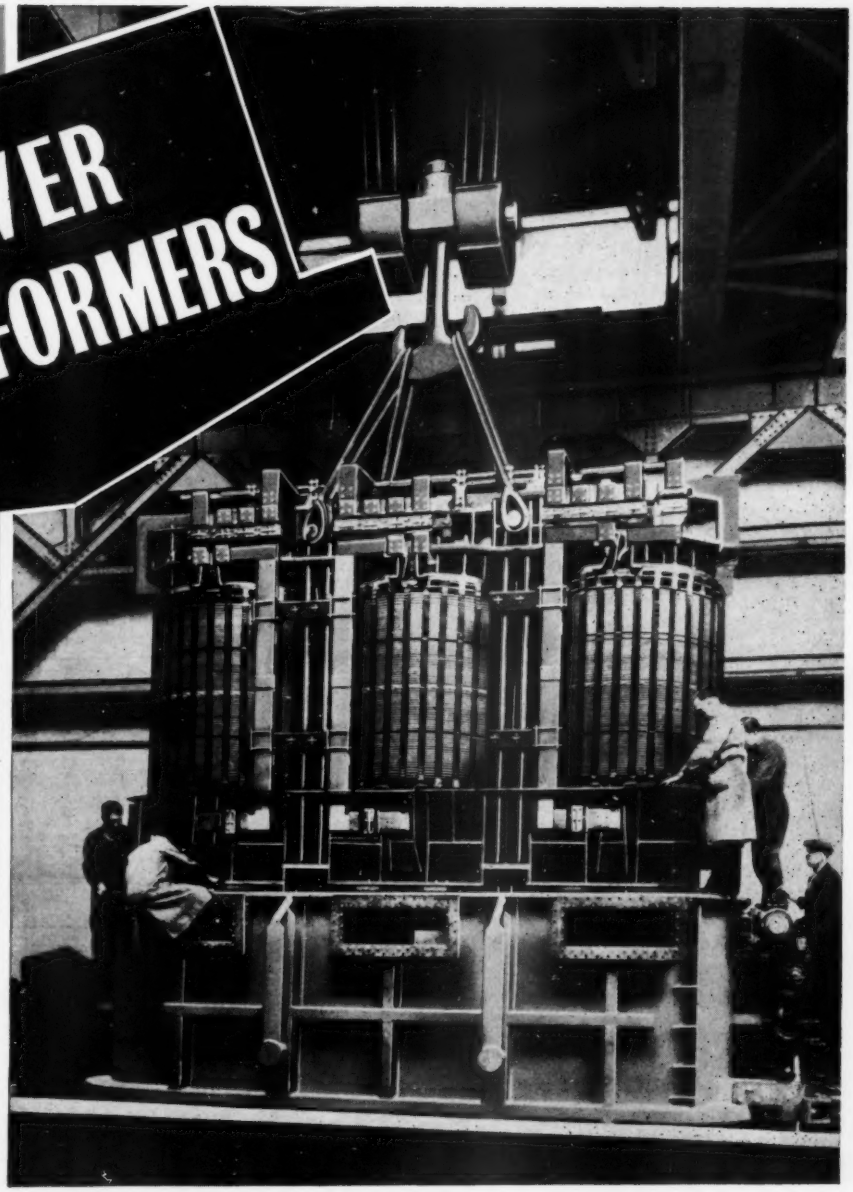


Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

GIOVANOLA **Monthey - Switzerland**

POWER TRANSFORMERS



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Bruce Peebles manufacture power transformers in the largest sizes for service at all voltages. Typical units in course of manufacture are 120,000 kVA 275,000-volt auto-transformers for the British Super-Grid.

Illustration shows a 75,000 kVA generator transformer in course of erection. It is typical of numbers ordered for direct connection to the generator to operate as a combined unit.

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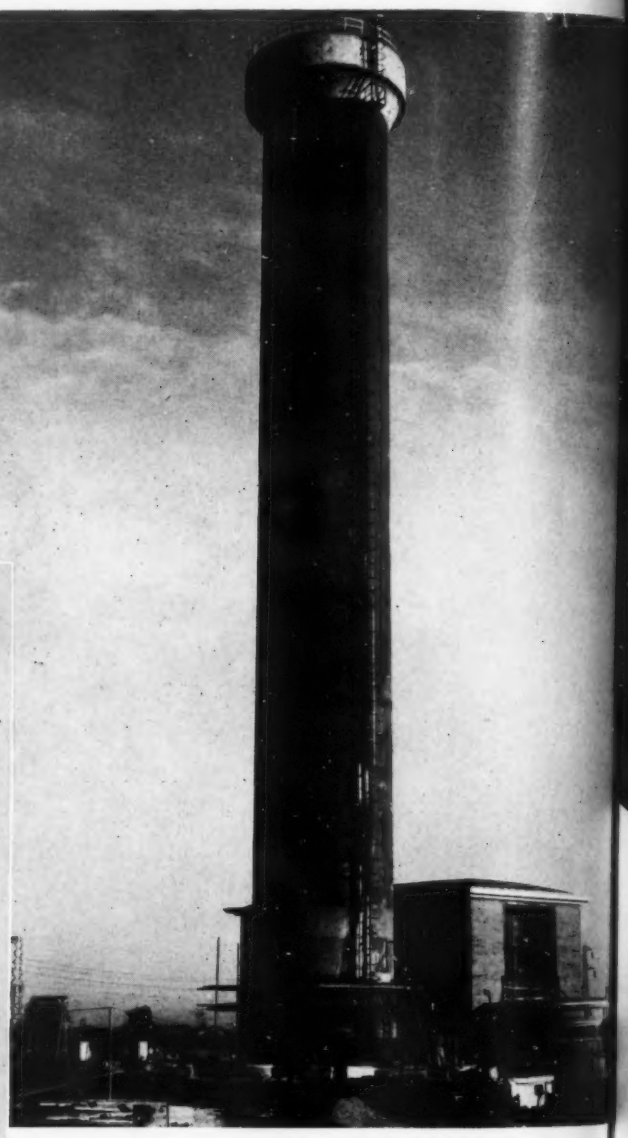

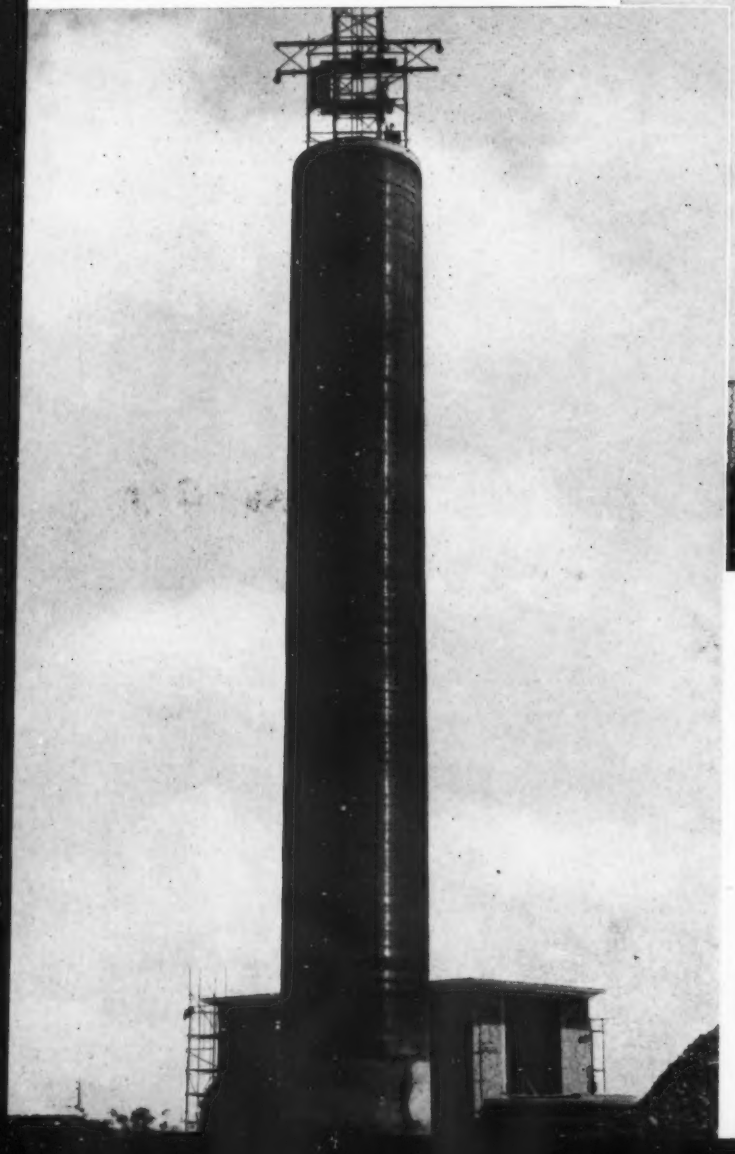

SURGE TOWER in prestressed reinforced concrete (Diameter 6 metres, Height 60 metres) for the MEDUNA Hydro-electric Station of

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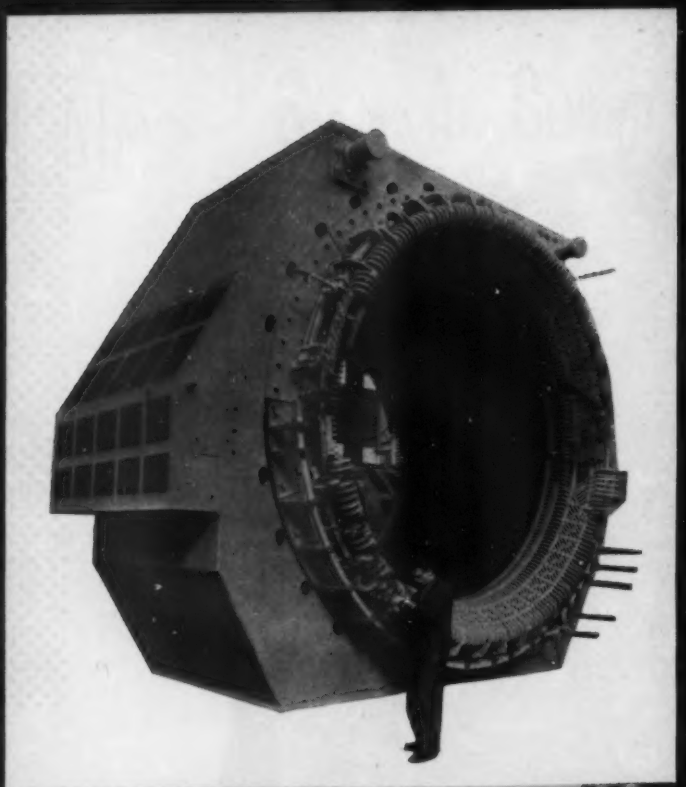


The stator of the 60MW hydro-alternator at Aurn Power Station, Norway.

Wealth o

G.E.C.

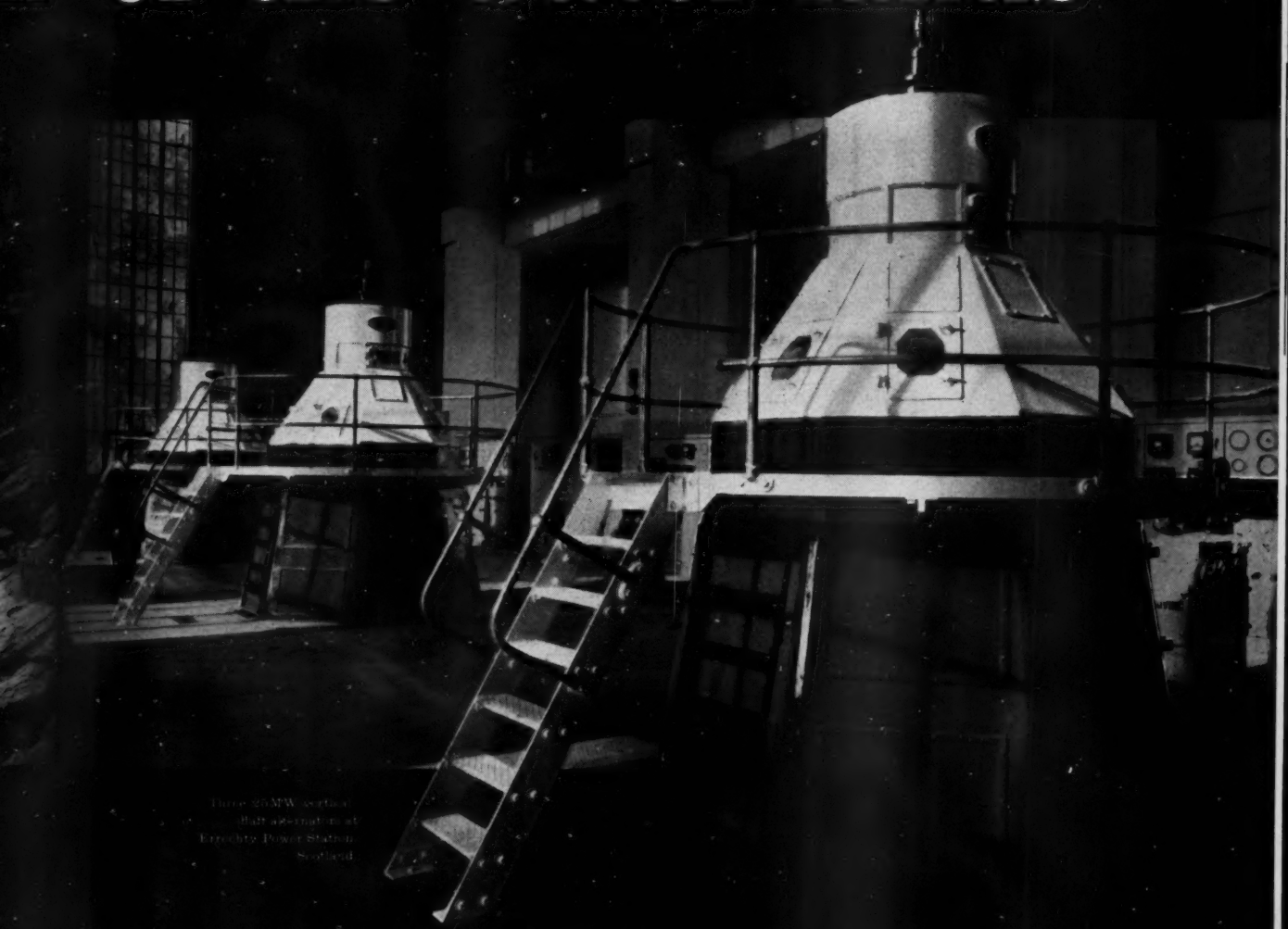
power

Wherever there's water in abundance, there's power potential in abundance too. In Scotland, the G.E.C. is helping to harness this power and has supplied the North of Scotland Hydro-electric Board with three 25MW hydro-alternators and control gear for the automatically controlled Errochty Power Station – the largest in the Tummel-Garry scheme.

Norway is another of the water-lands. Here, deep in the mountainside at the head of the Sunndalsfjord is the Aura Power Station. And here a 60MW hydro-alternator has been installed for the Norwegian State Power Company by the G.E.C. In Tasmania, too, the G.E.C. has recently installed five 25MW hydro-alternators at Tungatinah.

To Scotland . . . to Norway . . . to Tasmania . . . to the water-lands of the world, the G.E.C. sends hydro-electric equipment for generation, control and distribution of power.

h of the water-lands



Three 25MW vertical Hall alternators at Errochty Power Station, Scotland

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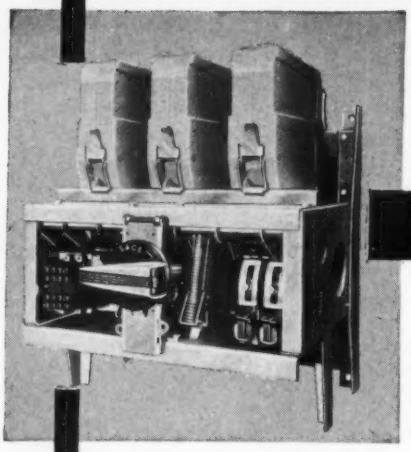

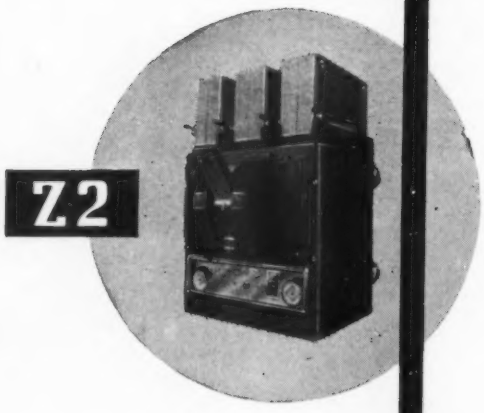

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Breaking capacity up to 20 kA
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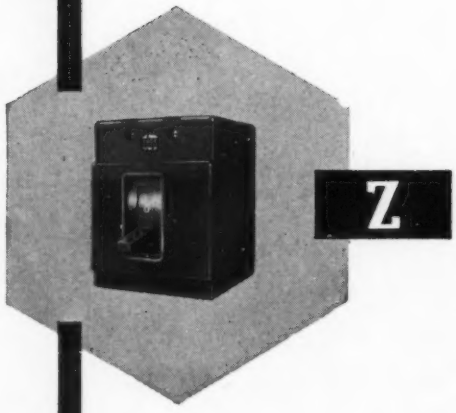

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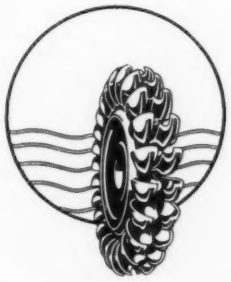

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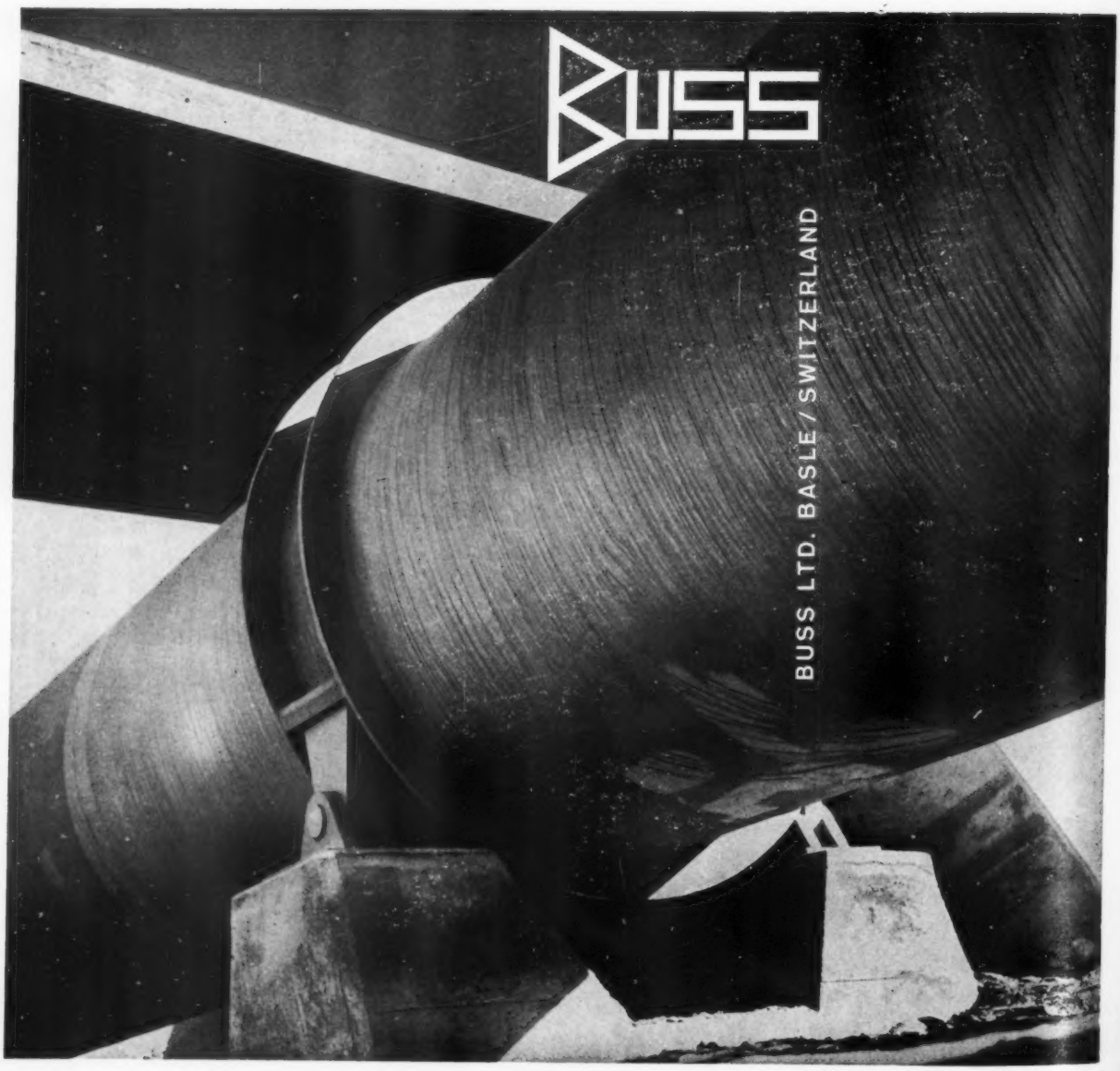

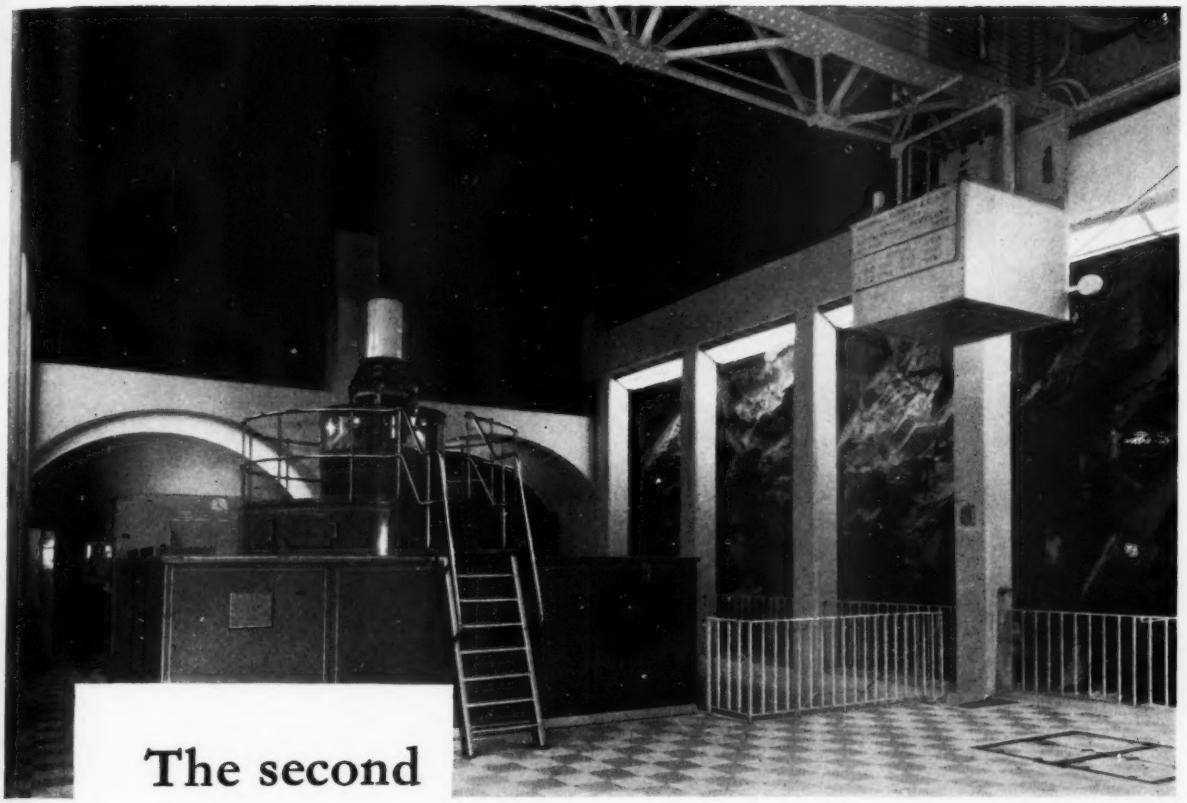

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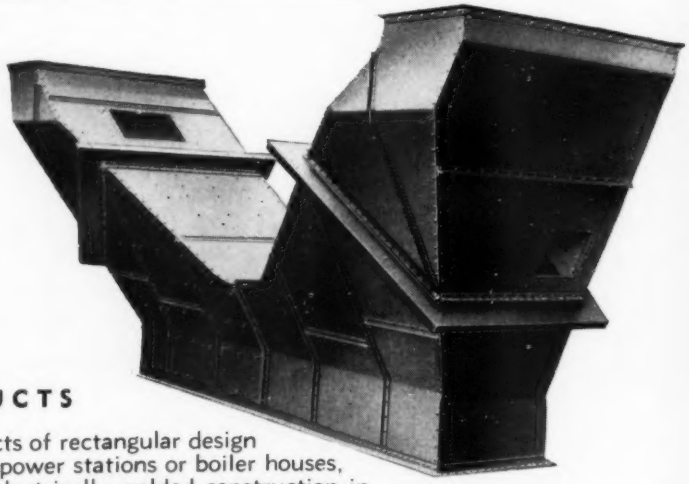

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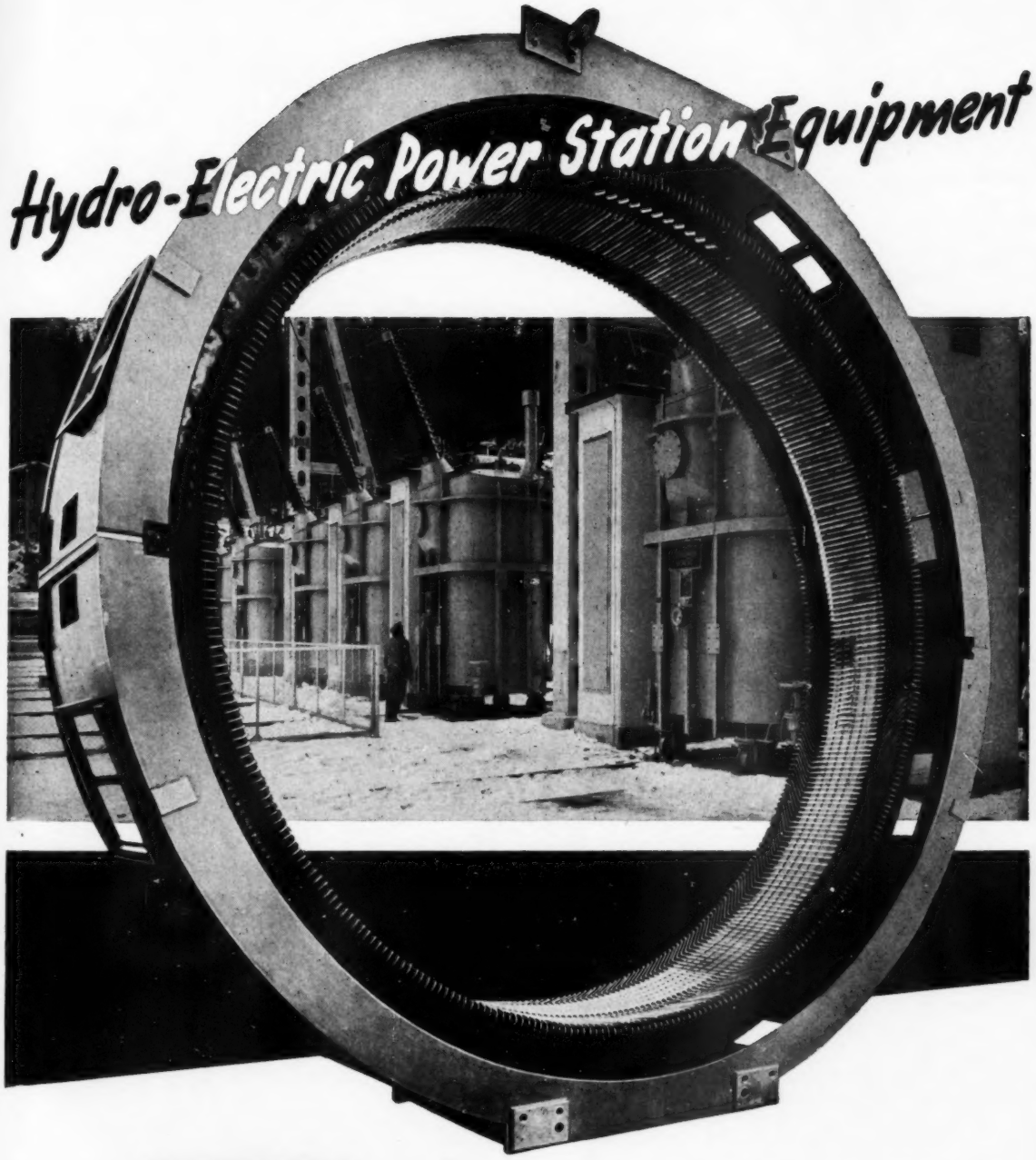

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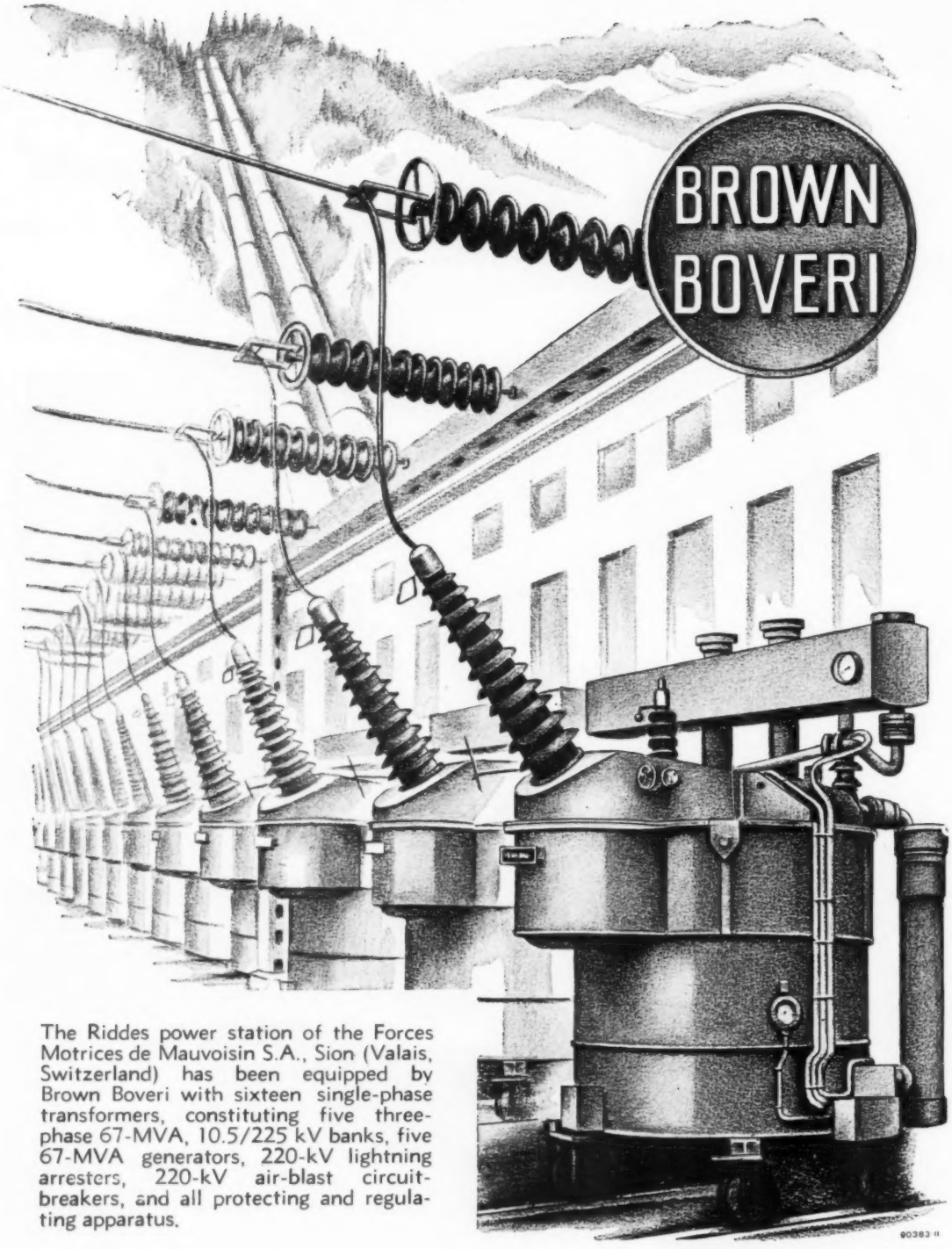

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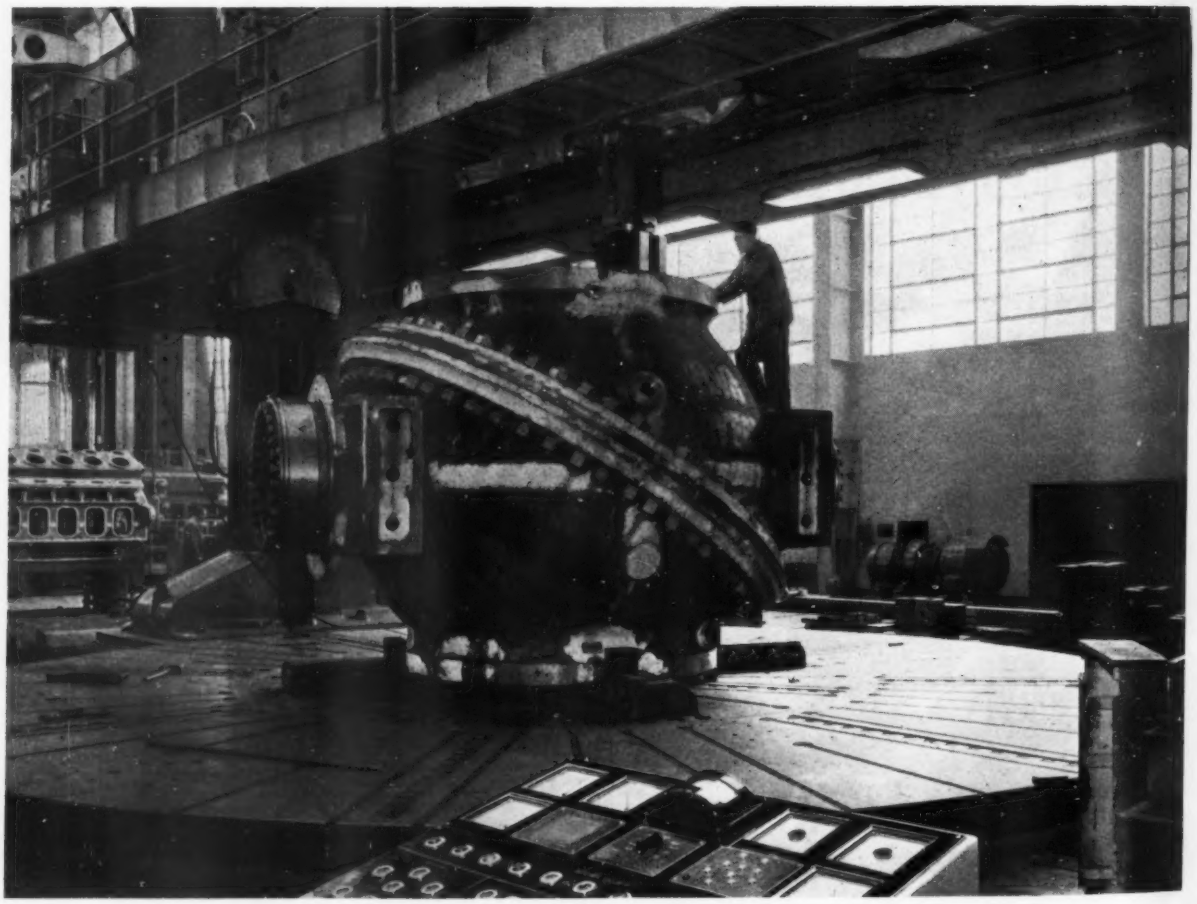

This picture shows a completely assembled Rotary Valve during its final machining stage

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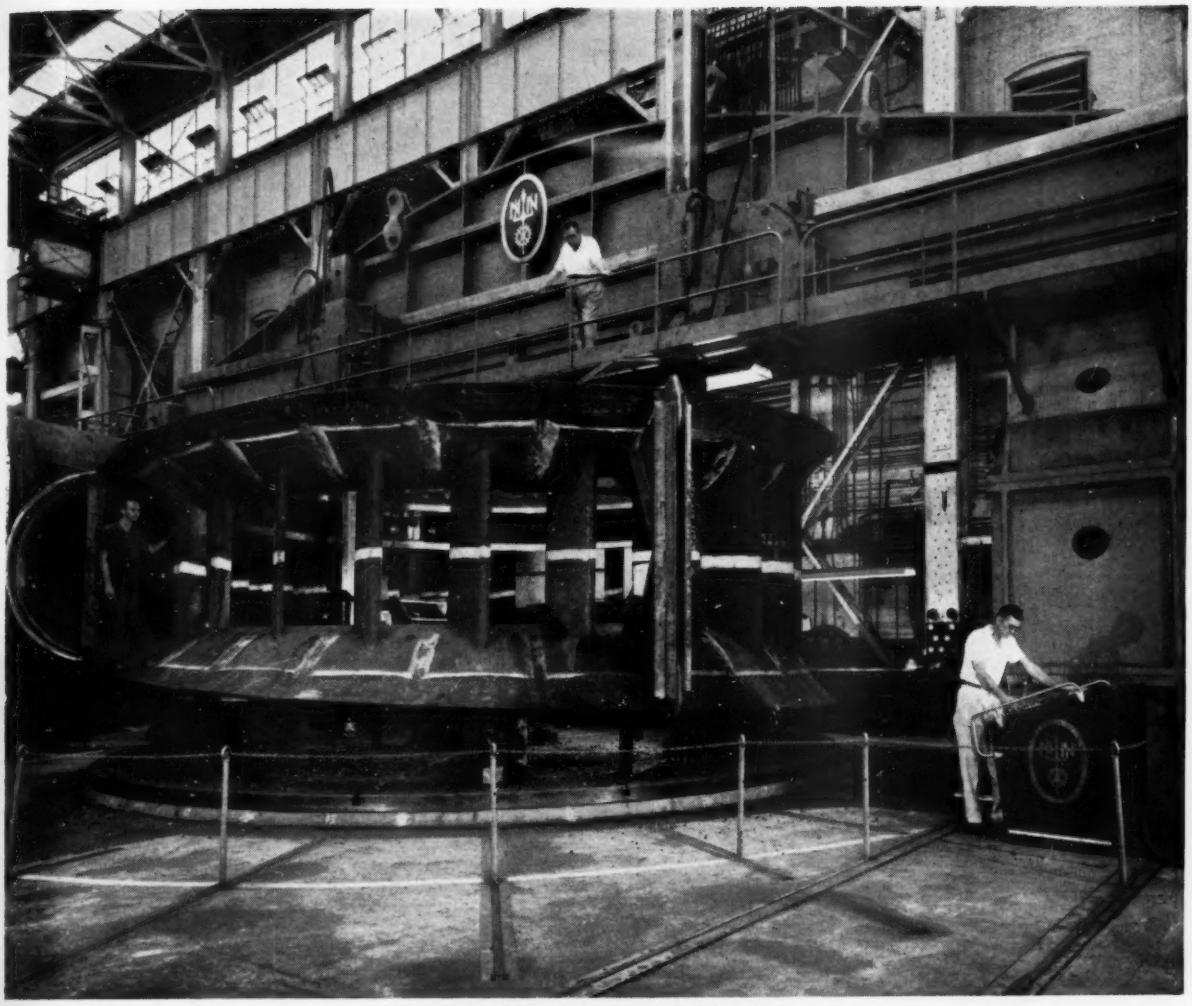

42 ft. boring mill-rough cuts a stay ring for one of the world's most powerful hydraulic turbines.

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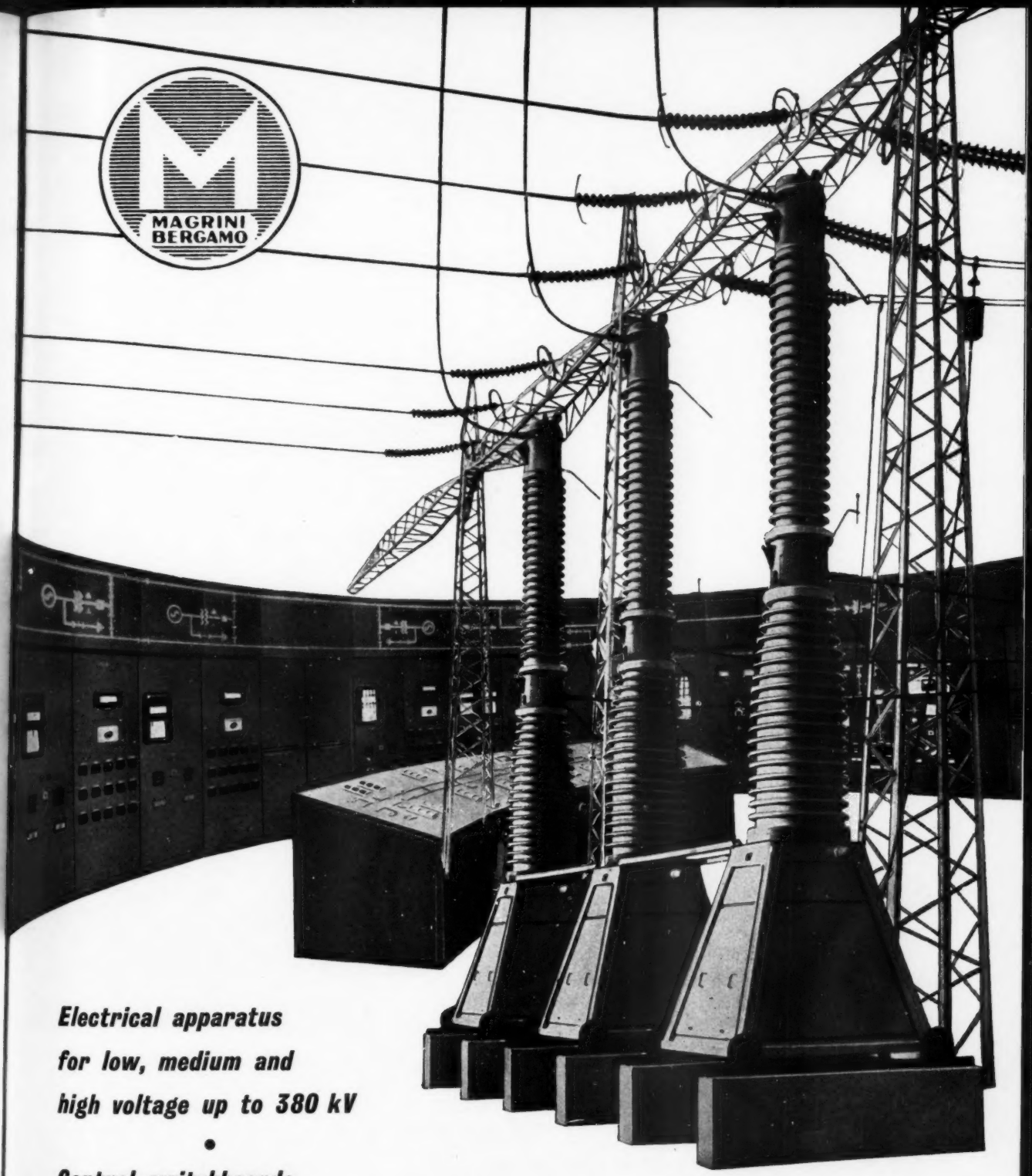

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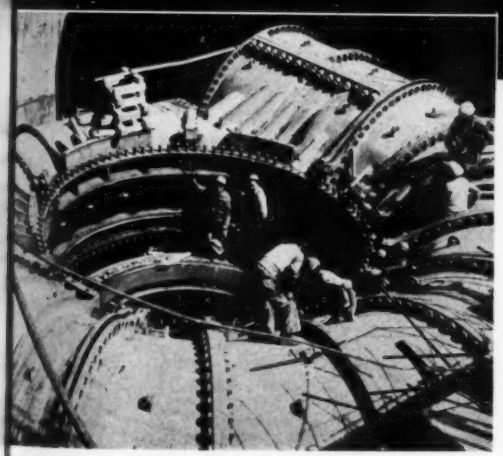

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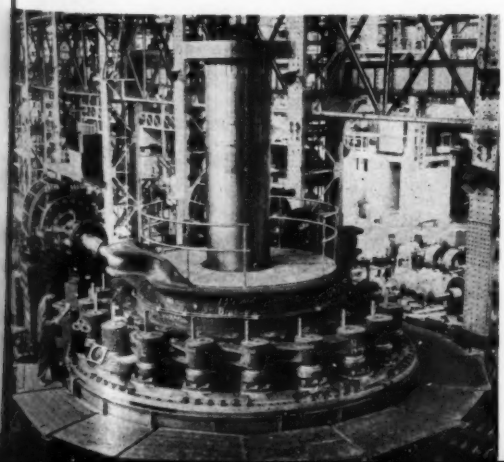

INDIA'S BHAKRA-NANGAL PROJECT INSTALLS **HITACHI** TURBINES

Bhakra-Nangal on the Sutlej River is the largest hydro project in India's First Five Year Plan and one of the largest multi-purpose river valley developments in the world. It provides flood control, irrigation to 10 million acres, and industrial power.

Two power stations at the 740-ft. high Bhakra dam will have ultimate installed capacity of 935,000 kW. At the left bank power station, the Government of Punjab is erecting 5 Hitachi vertical Francis turbines. Running at 167 rpm under a normal effective head of 400 ft., rated output of each unit is 150,000HP. Optimum performance will be maintained throughout severe head variations, ranging from 268 ft. to 512 ft.

High performance, the capacity for early delivery, and the sizable saving to the customer in initial capital outlay have given Hitachi a prominent position among the world's hydraulic equipment makers. Detailed information on any Hitachi hydraulic equipment will be sent on request.

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SHOP ASSEMBLY of Francis turbine for Bhakra power station. Hitachi-designed segment-type guide bearings and water seal where main shaft passes through head cover replace ordinary journal bearings and usual stuffing box, reducing maintenance.

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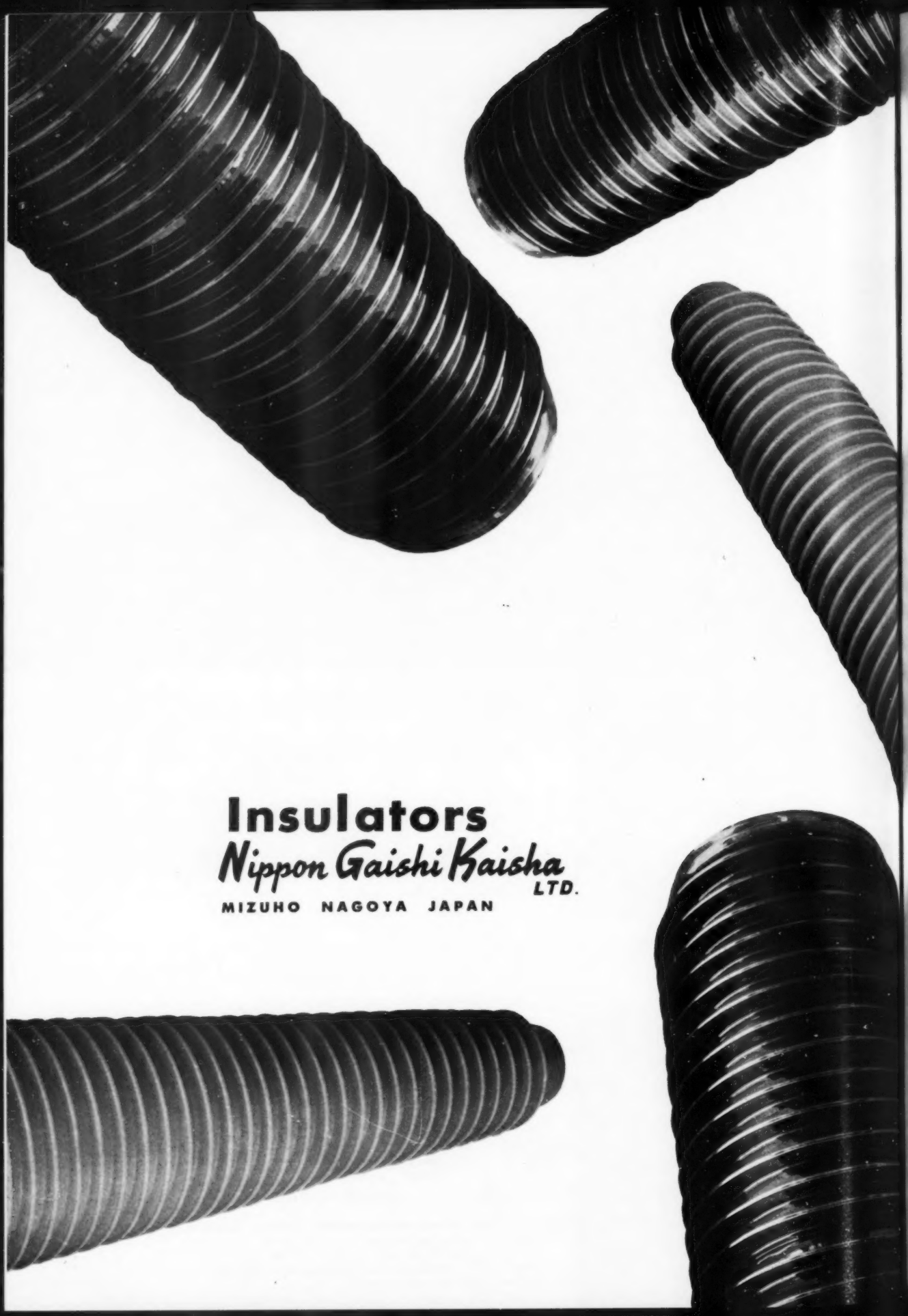
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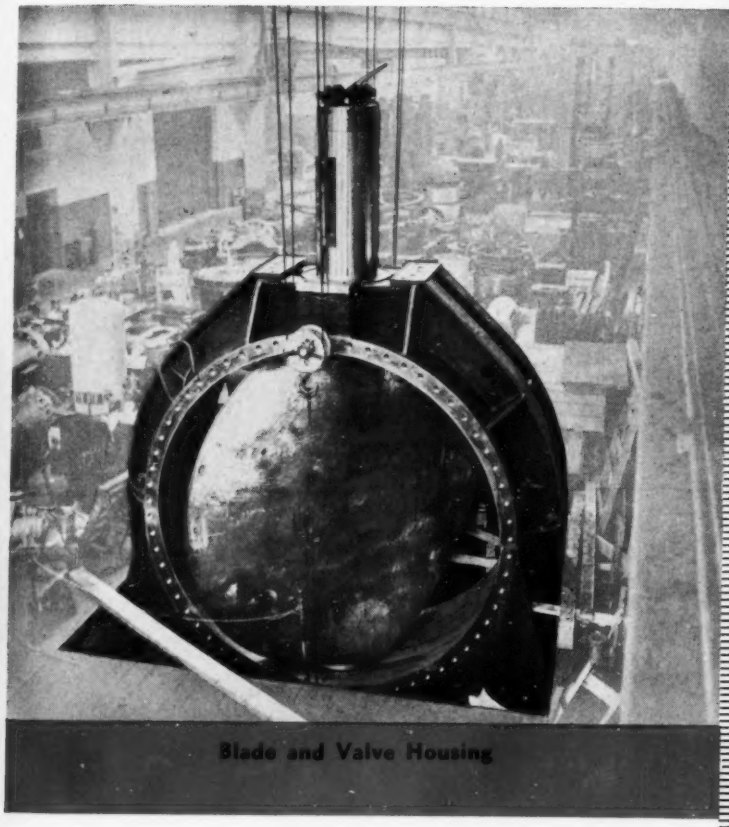

Blade and Valve Housing

Fitting the blade into the valve housing

The Butterfly Valve for one of the two KMW Francis turbines ordered by the well-known Swedish company Stora Kopparbergs Bergslags AB for their Trängslet Power Station is now under erection.

The complete valve weighs 90 tons and the valve blade alone 27 tons. The latter has a diameter of 4.5 m (14 ft. 9 in.). When the valve is shut, the water pressure on the blade will be 2,400 tons.

The first turbine is estimated to be put into service in October this year and the second unit about four months later. The total output of the two turbines will be 280,000 h.p. at 140 m (459 ft.) net head and 214 r.p.m.

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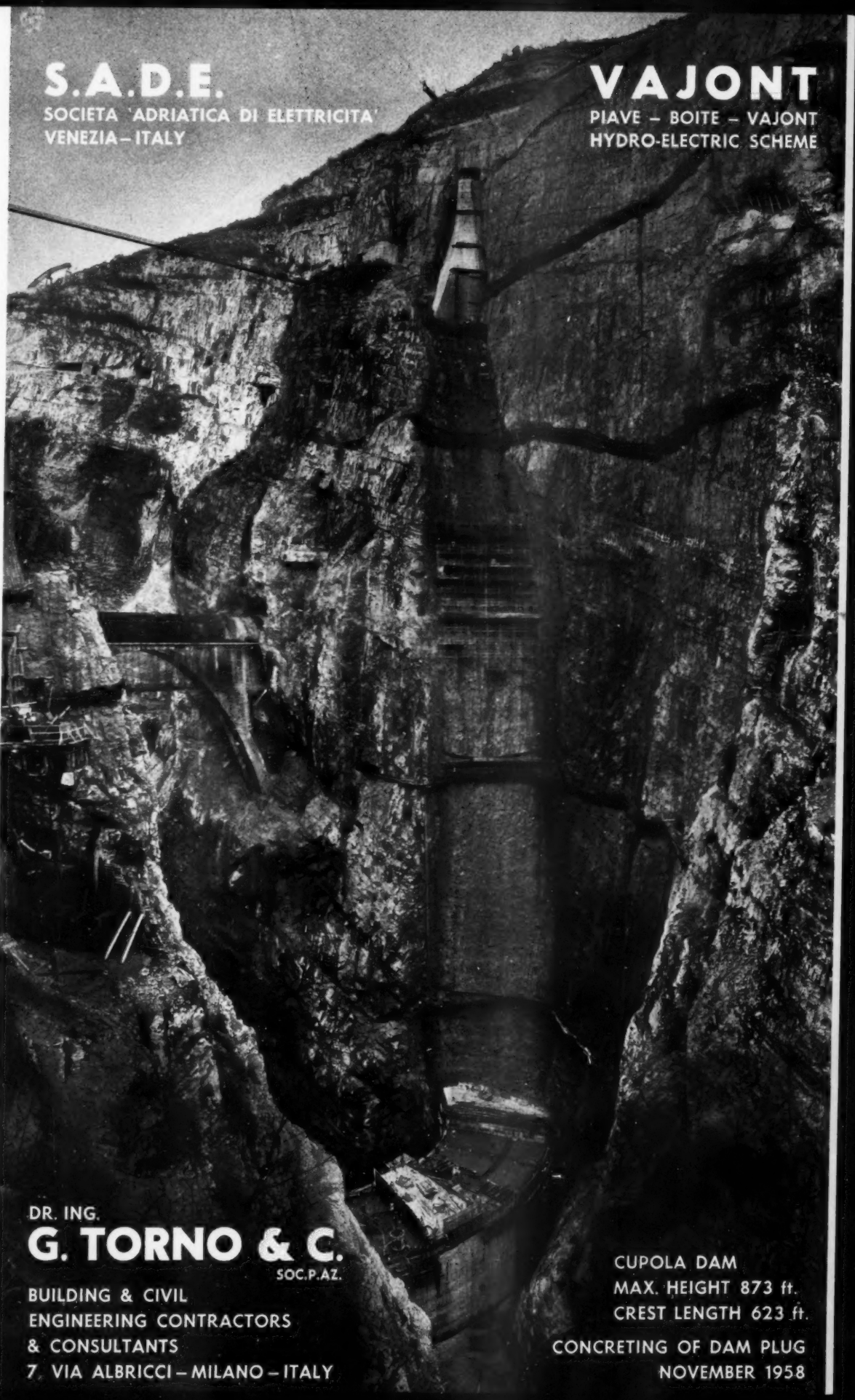
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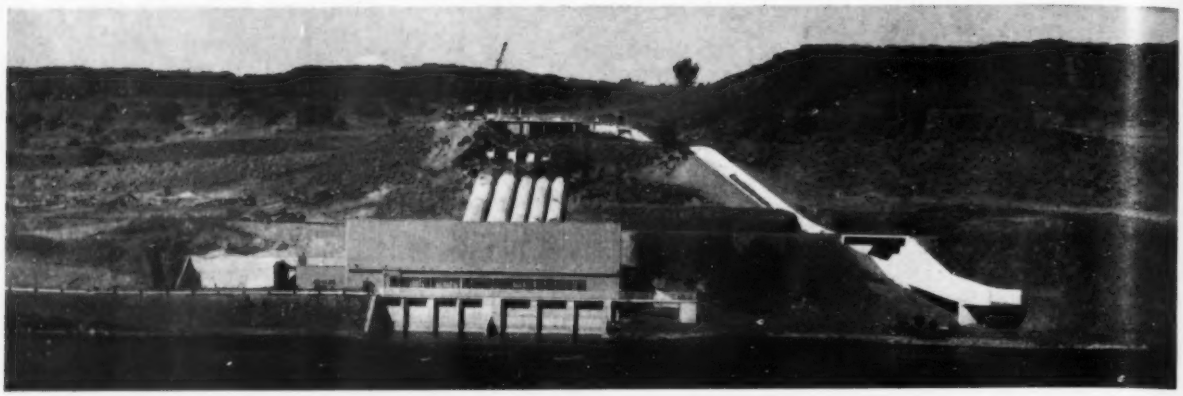

At the Bureau of Reclamation's Chandler Power and Pumping Plant, Leffel turbines drive both power and pumping units

A principal component of the United States Bureau of Reclamation's Kennewick Division of the Yakima project, the Chandler Power and Pumping Plant is situated near the junction of the Yakima, Snake and Columbia Rivers in south-central Washington.

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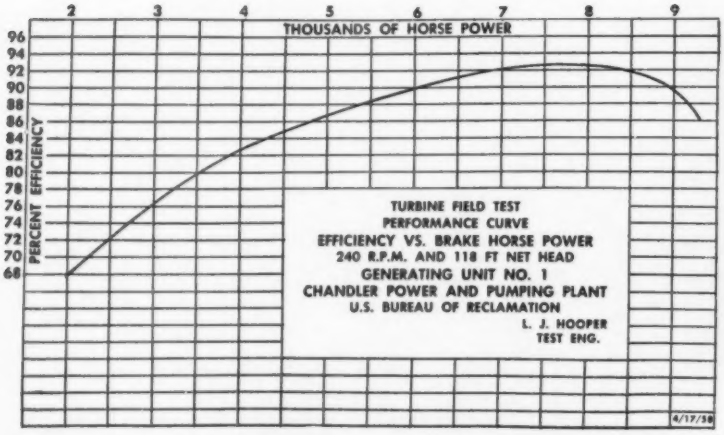

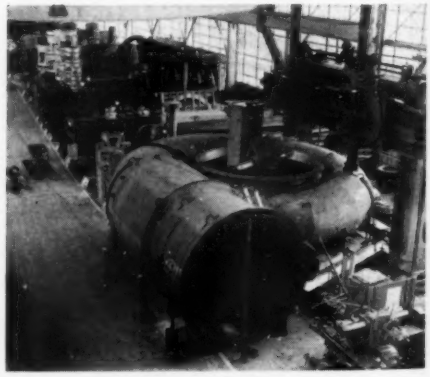

One of the spiral casings for the 8,500 HP turbines on Leffel's 24' boring mill.

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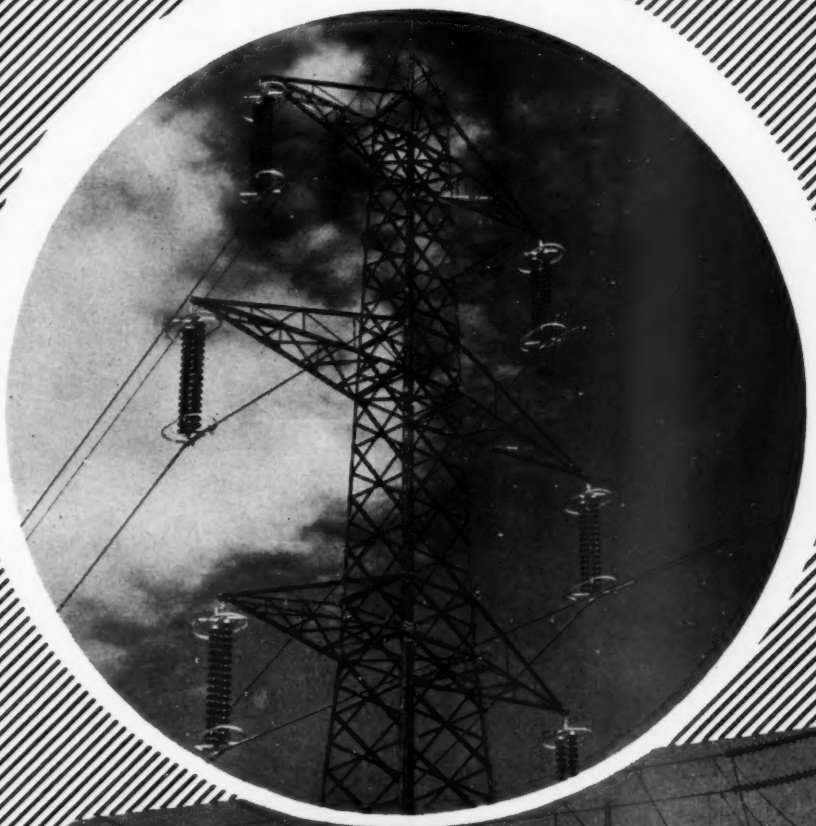

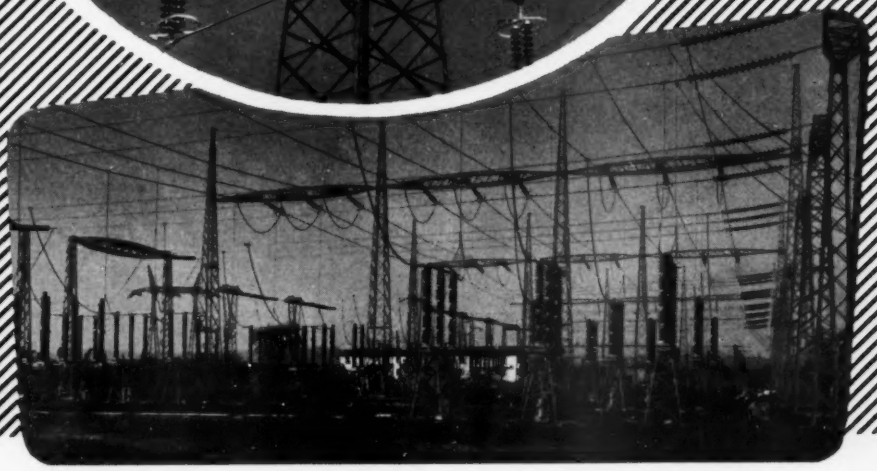

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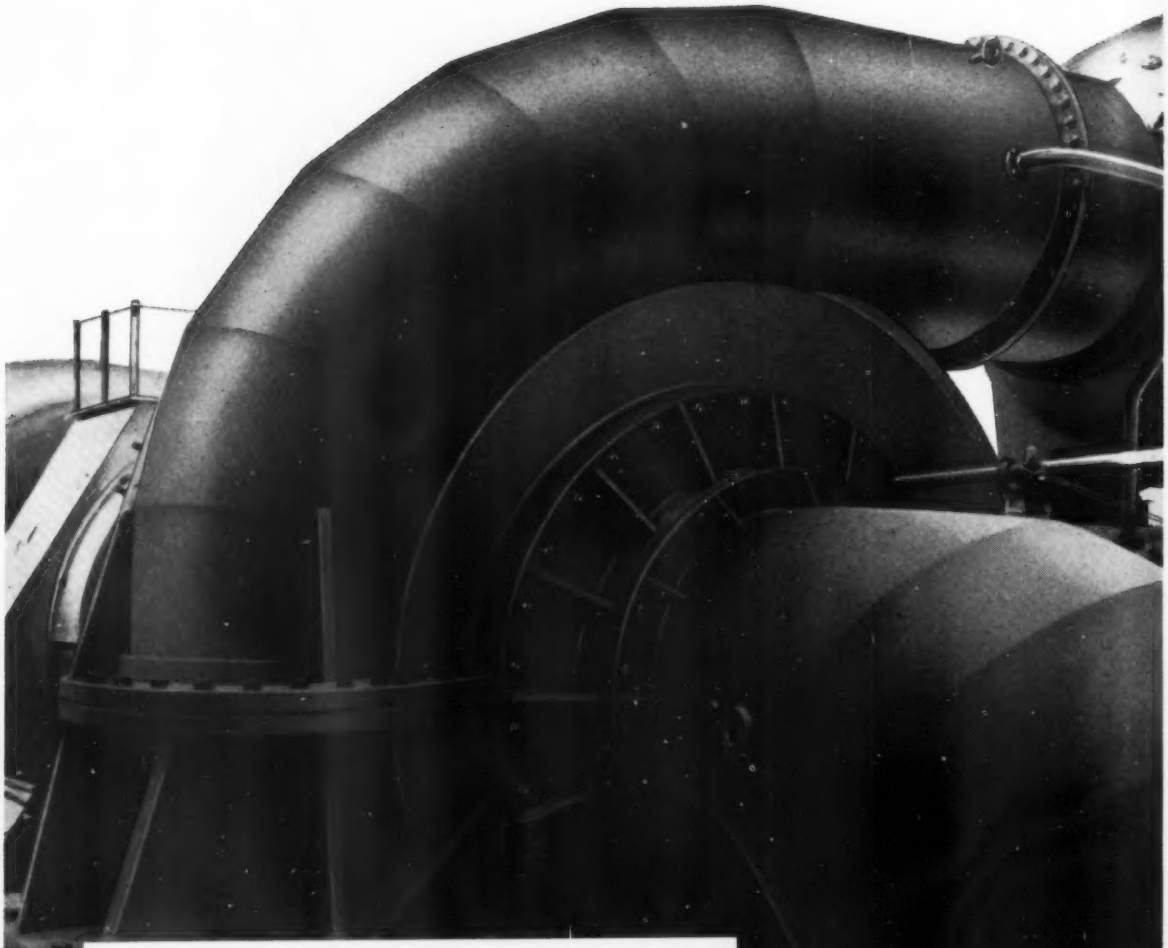
storage pump at Sron Mor



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New power for a Dominion from the Roxburgh Hydro-Electric Project

New Zealand's largest hydro-electrical undertaking, the Roxburgh project, will eventually feed an additional 320,000 kilowatts of electricity into the South Island grid system. The 1,200 ft. Dam, which is the largest ever built in the Dominion, was completed far ahead of schedule by Cubitts' New Zealand Co., in association with Conrad Zschokke Ltd., Geneva, and Downer & Co. Ltd., New Zealand.

This is one of many examples where the experience of the Cubitt Group of Companies is helping in the development of overseas territories.

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

The Hamilton Falls Scheme

ONE of the largest hydro-electric projects in the world is that for the development of the Hamilton River in Labrador, and on several occasions we have made brief reference to the investigations being carried out by the British Newfoundland Corporation Limited. About a couple of years ago an account of the preliminary survey was given in a paper presented to The Engineering Institute of Canada by G. V. Eckenfelder, W. P. Harland and E. N. Webb, but the published information has now been brought more up to date by a paper read recently by Mr. Webb before the Institution of Civil Engineers, London.* Mr. Webb, in his capacity as consultant to the British Newfoundland Corporation Limited, has written from an intimate knowledge of the investigational and preparatory work carried out on the scheme, and his paper is particularly opportune in that it gives a concise but lucid outline of the project. One is immediately impressed by the huge scale of the undertaking, for it is estimated that the Hamilton Falls can be developed to produce about four million firm horsepower in stages of one million horsepower each, and that a further two million can be developed below the Falls. Combined with this, however, is an unusually favourable combination of natural conditions, which predispose to an inherently simple and reliable installation with generating costs exceptionally low by any present-day standards.

The Hamilton River, with its two main tributaries the Ashuanipi and the Atikonak, has a catchment area of 21,000 sq. miles above the main falls and of 30,400 sq. miles above Muskrat Falls, 20 miles from Goose Bay. Above the main falls a high plateau of tough crystalline rock with a generally even but undulating surface of glacial·detritus, and with about 20% of lake area plus 10% of muskeg, affords an enormous supply of water with almost perfect facilities for regulation, at a general level of 1,500 ft. Local materials suitable for all forms of construction envisaged are in abundance. Hydrological data are somewhat limited in both extent and time, but taken in conjunction with much ionger records from comparable basins they are considered to indicate a minimum firm flow at Hamilton falls of 40,000 cusecs.

Of the various alternative methods of development considered, that proposed for adoption consists of extending Jacopie Lake down a natural channel by means of earth dykes to feed intake works above Portage Creek, where a head of 1,040 ft. can be utilised. The intake will be drowned to afford effective ice cover, and to guard against frazil ice the maximum velocity in the reservoir will be kept down to 2 ft. per sec. From the intake a vertical penstock shaft for each machine will lead to an underground power station, in which each of the four 1,000,000 h.p. stages will comprise four 200 MW Francis sets. The four machines of each stage will share a common free-flow tailrace tunnel of 42 ft. equivalent diameter and about 6,500 ft. long.

The projected layout, in fact, is reminiscent of Swedish practice, but on the scale on which it is envisaged it poses unusual problems of access, construction, ventilation, and electrical transmission. As regards electrical transmission it is proposed to adopt the unusual method of a double transformation. The first step from generator voltage to 132 kV will be effected underground and the connection to the surface will be by gas-filled cable. Thence an outdoor substation will raise the voltage to 400 kV for long-distance transmission, probably by means of auto-transformers.

Such a gigantic scheme, of course, cannot be constructed unless it is assured at the outset of a comparable block of load, and it is suggested that a metallurgical industry might be set up at Seven Islands on the north shore of the St. Lawrence, involving a transmission distance of about 250 miles. Based on 1956 figures the price of an initial block of 1,000,000 h.p. delivered to Seven Islands would be of the order of $25 per h.p. year, of 2,000,000 h.p.

* "Hamilton Falls Hydro-Electric Scheme." Paper No. 6295. By Eric Norman Webb, D.S.O., M.C., M.I.C.E. Institution of Civil Engineers, Great George Street, Westminster, London, S.W.1.

$20, and upwards of 2,000,000 h.p. somewhat less than $20. This means roughly 4, 3¼ and 3 mils per kWh. It is also considered to be technically and economically feasible to transmit a large block of power some 1,000 miles to the power-hungry regions of Quebec, Ontario, or the Maritime Provinces.

In an interesting discussion Mr. T. A. L. Paton (of Sir Alexander Gibb & Partners) and others referred to the importance of timing in carrying out a scheme of this size, and raised the question of the probable economics of the scheme vis-a-vis correspondingly large schemes in Western Canada. Mr. J. K. C. Hunter (consultant to Sir Alexander Gibb & Partners) entered a caveat as to the possible impact of nuclear power and questioned the adequacy of the hydrological data. Mr. Webb seemed to have complete confidence in the reliability of the preliminary investigations, which had, in fact, covered all the points raised, and appeared satisfied that the estimates, although liable to modification, were unlikely to be jeopardised by constructional difficulties, manpower shortages, or other major contingencies.

On the constructional side, Dr. Charles Jaeger referred to the exceptionally favourable conditions for development by means of vertical penstock shafts and free-flow tailrace tunnel, and Mr. F. F. Ferguson suggested the use of a precompressed concrete shaft lining in place of the steel lining proposed. Mr. A. Wilmot (of the English Electric Co. Ltd.) painted a reassuring picture of the feasibility of building the 200 MW generating sets, which, of course, would be the largest of this type yet constructed. Mr. R. T. Gerrard (of Messrs. Binnie, Deacon & Gourley) gave an interesting account of a development constructed in 1954 on Lake Menihek, 100 miles north of Hamilton Falls, and mentioned a difficulty that had been encountered of tannic-acid contamination of the local aggregate for concrete—a difficulty, Mr. Webb assured him, that did not arise with the aggregate selected for Hamilton Falls. Mr. L. B. Ratcliffe (of Messrs. Kennedy & Donkin) considered that to transmit a large block of power 1,000 miles to Ontario would form an ideal case for high-voltage direct current and would be economic on this basis.

With a scheme of this magnitude there is a tremendous amount of detail to be filled in, but Mr. Webb's paper admirably picks out the highlights and reveals the essential simplicity of a project that may prove of the utmost value to Eastern Canada.

Power from Fraser River

A REPORT issued jointly by the Canadian Government and the Province of British Columbia argues that it is practicable to construct a large power system on the Fraser River without affecting to any considerable extent the extraordinary salmon runs which sustain the canning industry. The present preliminary scheme, as announced by the Fraser River Board recently, would be confined to the upper reaches of the Fraser and its tributaries by a series of small dams which would not affect the lower spawning grounds, which indeed would benefit from better regulated incubation conditions and absence of destructive floods. Only later when the riparian life of the salmon was more fully understood and could therefore be fully protected in the new conditions, would any attempt be made to harness the main Fraser stream for power purposes. With the preliminary scheme confined to the headwaters it is claimed that something like 1¼ million horsepower could be developed. It is reckoned that this could be provided more cheaply than power obtained from either the Peace River or the Columbia River developments.

Turbines for Vargfors

THE KMW concern have been awarded the order for a Kaplan turbine to go into the Vargfors power station in the North of Sweden. The customer is Kungl. Vattenfallsstyrelsen (The Swedish State Power Board). This plant will be situated on the Skellefte river, where all the major power stations completed or under construction are equipped with KMW machines. These stations are the Selfors, Krångfors, Granfors, Finnfors, Båtfors and Kvistforsen.

The new station at Vargfors will be equipped with a vertical Kaplan turbine of 82,800 h.p. at 157 ft. (48 m.) head and 150 r.p.m. With a head of 162 ft. (49·3 m.) the output is increased to about 100,000 h.p. The turbine will be of all-welded design and have a spiral casing of steel plate. The station is expected to be put into service on October 1, 1961.

Dam Burst in Spain

OFFICIAL news as to why the dam on the Tera River should have burst in North-west Spain recently is still lacking, and no doubt a full technical investigation will be made before anything is published. As the burst occurred during the night the death roll was lamentably heavy and 132 persons have been reported as being dead or missing.

The dam was of the buttress type constructed in masonry. The centre section, which contained the spillway, was swept away together with the majority of the buttresses and the power house and village of Ribadelago were quickly overwhelmed. The dam was 112 ft. high and was owned by the Moncabril Hydro-Electric Company, who also installed the associated 43 MW power station.

Snowy Mountains Annual Report

THE Annual Report of the Snowy Mountains Hydro-Electric Authority for the year ended June 30, 1957, has been published recently. There was great progress during the year under review, including completion of the preliminary investigations, contract designs and specifications for the 280,000 kW T.2 project, the Tooma-Tumut diversion and the Murrumbidgee-Eucumbene diversion.

The Guthega power station produced less power than during the preceding twelve months because the summer and winter seasons were drier than usual; for the 1956/7 period, 180,755,800 kWh were generated, of which 177,564,545 kWh were supplied to the grid at an approximate cost of 0·91 pence per kWh.

Constructional activities during the year were mainly concentrated on Adaminaby dam on the Eucumbene river and on the Upper Tumut project, including the 14-mile Eucumbene -Tumut tunnel, the Tumut Pond dam and the T.1 pressure tunnel, underground power station and tailrace tunnel. On the basis

of progress achieved up to June 30, 1957, it is anticipated that the two 80,000 kW turbo-generators in T.1 power station, operating on the discharge of the Tumut river without the Adaminaby reservoir storage capacity, will come into service in 1959. The remaining two units in the power station will commence operating at the end of 1959 or early in 1960, by which time Adaminaby reservoir and the Eucumbene-Tumut tunnel will be in service.

The T.2 project, comprising a concrete dam, seven miles of tunnel and a 280,000 kW underground power station, is located some six miles downstream of T.1. The power station will be located in granite formation some 800 ft. underground. During the year, an exploratory tunnel was driven to the site of the station and sufficient data were obtained to fix the final location of the station.

The proposals for the Snowy-Murray development recommended by the original Snowy River Committee provided for a storage of 1,100,000 acre ft. at Jindabyne and a 30-mile tunnel through the main Dividing Range to the Swampy Plains river. They also included the diversion of the Upper Snowy river, the Geehi river, and Bogong Creek into the main diversion tunnel where it passes under these streams. A careful study of the data obtained by the Authority's investigation forces has revealed that some improvements can probably be made to the original proposals for the Snowy-Murray development, including a rearrangement of the tunnel system and the possible use of Francis turbines.

Despite some initial delays, and a recurring shortage of qualified personnel, progress has been good on the whole Snowy Mountains scheme and the schedule of work has been maintained.

New Zealand Power Restrictions Ended

ALL power restrictions in New Zealand ended last December. After 17 years of restrictions the Government announced their cessation and hinted that, with care by all, the same burden of restrictions might never again be necessary. New generating stations brought into operation at Meremere, Wairakei and Atiamuri have provided the Government and the electricity authorities with the equipment to supply sufficient power, but over the last few months of 1958 water storage was at too low an ebb to dispense with the restrictions. Heavy rain in the two main North Island watersheds of Lakes Taupo and Waikaremoana towards the end of the year, however, built up the "hydraulic bank account" to the required level.

Belgian Transformers for Sweden

THE Swedish State Power Board has ordered four 220 MVA single-phase transformers from ACEC, Charleroi, to form a 660 MVA three-phase bank for the Vargfors station on the Skellefta River, the fourth unit being spare. Vargfors station will be of the underground type, and the transformers will be in an underground chamber.

The transformers will be of the shell type, and each unit will have four windings designed for 400/149·5/13·7/13·7 kV. The presence of two primary windings is explained by the fact that the bank will be supplied by two 70 MW generators, reactive power being furnished by 80 MVAR synchronous condensers. Each unit fully equipped will weigh 210 tons but can be stripped down to 160 tons for transport.

It is believed by the manufacturers that this transformer bank will be the second largest in the world. ACEC has just completed the 330 kV transformer substation for one of the Snowy Mountains power stations, and in this high-voltage high-power class of equipment has also built the 400 kV 3×105 MVA transformers for the Pikkarala substation serving the Finnish Grid.

Tunnelling Completed Nine Months Ahead of Schedule

AN outstanding achievement in the North of Scotland Hydro-Electric Board's programme has been confirmed with the announcement that the driving of the network of tunnels in the Killin (Perthshire) section of the Breadalbane Scheme has been completed. Work began on this section on April 1, 1957, and the driving of the tunnels has been completed nine months ahead of the Board's schedule. To do this, progress has been maintained at rates which it is believed have created new precedents in the hydro-electric field.

The project involved the main contractors, The Mitchell Construction Co. Ltd., in the building of 18 miles of access roads, 14½ miles of tunnels and seven miles of pipe aqueducts up to 6 ft. diameter. The work is split over an area of about 50 sq. miles and required survey work over mountains more than 3,000 ft. high. By using a helicopter, Mitchell's speeded up the surveying sufficiently to allow tunnelling to start on June 1, 1957, since when over 3½ million feet of blast holes have been drilled and over 1,000,000 lb. of explosives used in excavating the 14½ miles of tunnels.

The high speed of tunnel driving which the contractors have attained on this work is over three-quarters of a mile per month and equal to their achievement on the St. Fillans section of this scheme, where they claimed a world record of 557 ft. of tunnel driven in one week on one heading.

B.C. Electric's 1959 Programme

DURING 1959 B.C. Electric is carrying out a $95,800,000 expansion programme, more than 80% of which will be devoted to new electrical facilities. The biggest single expenditure during the year will be $22,400,000 for further work on the second-stage development of the Bridge River hydro-electric project, which will provide an additional 345,000 h.p. before 1961 to meet growing power demands of the Lower Mainland, Southern Vancouver Island and portions of the interior of the province. The total cost will be $56,500,000.

Another $18,100,000 will be spent in 1959 on the Burrard steam plant, which will have a generating capacity of 800,000 h.p. from four generating units before 1965.

The world's largest gas-turbine electric generating plant at Port Mann, to be used eventually to firm up the hydro capacity, will be completed in 1959. Expenditure during the year on this 134,000 h.p. plant will be $2,100,000.

Continued work on new mainland transmission

lines to bring electric power to service areas will cost $11,700,000. More than $5,000,000 will be spent to construct or enlarge mainland substations required to reduce transmission line power to lower voltage. A further $5,700,000 will be spent on the mainland during the year on distribution lines. Work on Southern Vancouver Island generating stations, substations, transmission and distribution lines is expected to cost $3,200,000. A further $10,000,000 will be spent for line and service extensions on both the Mainland and the Island. Total expenditures during the year on new electrical facilities will be $79,000,000.

New Power Station on Waikato River

A NEW 60 MW station has been planned for the North Island, New Zealand, by the Ministry of Works and Electricity Department. The plant will occupy a site on the Waikato River, which has long been a tourist attraction as the rapids are very impressive from a scenic point of view. Some doubts have been expressed as to whether the new power station would destroy this attraction but the authorities have been able to give assurances that this would not be so and have had a model built to fortify their assurances. This model measures 28 ft. by 8 ft. and details about half a mile of the gorge on a scale 1 : 200 horizontally and 1 : 100 vertically. It has been constructed by Mr. R. C. French and Mr. D. W. Hefford, both of whom are architectural modellers. The dam will only be 15 ft. in height and there will be no pipeline as the water will be carried down to the turbines by means of a tunnel. Neither the surge chamber nor the power house will be apparent from the popular lookout points and the lake can scarcely be said to interfere seriously with the natural scenery. The scheme will have the advantage of causing a first-class roadway to be built to the rapids.

Power Supply in Sweden

A REPORT on Swedish power supply for the year July 1, 1957, to June 30, 1958, has been issued by CDL (Centrala Driftledningen) and a copy has been sent to us by the Swedish Water Power Association. Precipitation during the year was 111% of the average, and in central and southern Sweden the snow coverage, which mainly determines the size of the spring flood, was much more extended than usual. In consequence the output of hydro-electric power was 17% greater than in the previous year, whereas thermal power production was reduced by about half. The trade recession prevailing in the period under review is reflected in the fact that prime consumption of firm electric power rose by only 5·6%, which was lower than had been expected. Industry used only 4·4% more power than in the previous year, whereas supplies to private consumers rose by 8·8%, or about the average rise over the latest 10-year period. Power supplied to tramways and railways was somewhat lower than previously. On the other hand considerable increases were recorded in consumption for electric steam boilers and pumping (1,220 GWh), and in power exported to Denmark, which amounted to 555 GWh.

During the year under review work was started on extensions to hydro-electric capacity amounting to 685 MW, corresponding to an average annual production of 2,825 GWh. Hydro-electric plant completed during the year amounted to 480 MW, bringing the total installed hydro-electric capacity in Sweden to 5,770 MW. This is capable of an average annual production of 28,770 GWh.

Proposals for Dominican Republic Scheme

THE Dominican Republic is inviting overseas firms to submit proposals for a hydro-electric project on the river Nizao, in the south of the Republic. The proposed dam will store 600 million cu. m. of water and the power plant will have a capacity of 135 MW. Combined with the hydro-electric scheme will be an extension of the aqueduct for the capital, Ciudad Trujillo, which is designed to meet the estimated needs of the city over the next twenty years. The plan also makes provision for a water purification plant at San Cristobal. The irrigation canal system in the Bani area is to be considerably extended.

Further details of the scheme may be obtained from the Oficina del Ingeniero Asesor del Poder Ejecutivo, Ciudad Trujillo, Dominican Republic.

Selling Price of Electricity

A PUBLICATION issued by The Organisation for European Economic Co-operation discusses "The Trend of the Selling Price of Electricity and its Relation to the Financing of New Plant," this being the title of the report. Both industrial and household consumption of electrical current has been rising steadily and the changes of rates that have been necessary do not appear to have had any appreciable effect on economic activity or living standards. Expansion of supply is still an urgent matter and some increase in the cost of electricity may be expected in view of the increasingly higher costs that are involved for mining deeper and more difficult coal and the heavier capital outlays that are needed for nuclear generating stations. The difficulty of raising further capital has been felt in member countries where electricity undertakings are unable to sell current at a price which covers not only production costs but also a fair return on the capital invested. The possibilities of giving investment a normal yield and of building up adequate depreciation reserves are, the report states, the most effective means of attracting private capital and securing an expanding industry. The report is obtainable at the O.E.E.C. headquarters in Paris at a price of 10s. ($2·00, 600 Fr. fr., or 6·70 Sw. fr.).

Hydro-Electric Progress in Canada, 1958

UNDER the authority of Honourable Alvin Hamilton, Minister of Northern Affairs and National Resources, the Water Resources Branch has issued Bulletin No. 2625 which gives an annual review of current water-power development throughout Canada, and of other construction pertaining to electrical power and supply and distribution.

A record was established in 1958 in the amount of new hydro-electric generating capacity brought into service during a period of one year. A net

total of 2,485,040 h.p. of new capacity was installed, after making allowance for the dismantling of 6,360 h.p. of existing capacity. The previous high record year was 1954 during which 1,758,450 h.p. of new capacity was completed. Ontario's total of 1,301,800 h.p. of new capacity installed during 1958 was the highest among the provinces, followed by Quebec which added 900,000 h.p. The total installed capacity of water power plants in Canada is now listed at 22,376,048 h.p. In addition to installations completed in 1958, new developments or additions to existing plants are expected to add about 2,000,000 h.p. of new capacity during 1959.

The building of new thermal-electric plants or additions to existing plants is increasing in some areas of the country; the construction of main transmission lines, distribution lines and substation capacity also proceeded vigorously during the year.

Copies of Bulletin No. 2625 may be obtained free of charge from the Director, Water Resources Branch, Department of Northern Affairs and National Resources, Ottawa.

Correspondence

Bolted Angle Struts

To the Editor, WATER POWER

In his article on this subject in the January issue, Mr. Gregory's opening paragraphs would have been clearer had he made a distinction between standard high-tensile bolts and high-tensile "friction-grip" bolts. The former have been employed for many years on tower work in conjunction with the growing use of high-tensile steel members, and they are manufactured and function in a similar way to mild steel bolts, but with higher allowable stresses. On site the nuts on both mild and high-tensile bolts are fitted not much more than finger tight during tower assembly, and then finally tightened with standard length spanners. Providing they are sufficiently tight it is a mistake to over-tighten, as this can either strip the threads, or even twist off the nuts of small diameter bolts, and the strength of the bolt depends on its shear and bearing value.

High-tensile friction-grip bolts function as the name implies, and in the manner described by Mr. Gregory, and no doubt they have a widening field of application in general structural work. However, for well known reasons 95% of transmission tower steelwork is galvanised and this prohibits the use of friction-grip bolts because the zinc deposited on the surface acts as a lubricant in the sense of reducing the friction value between the contacting surfaces. Obviously manufacturers of friction-grip bolts cannot give any guarantee of performance in such circumstances, and as the friction-grip bolt is tensioned up to nearly yield value it is incapable of taking full shear loads in the event of slip. Therefore, apart from extra cost, there would be a positive disadvantage from their use, and as far as transmission towers are concerned the question as to what extent the "grip" value might enhance the end fixing value of angle struts becomes purely academical.

Experienced tower designers will be the first to admit that there is still a lot to be learnt about the behaviour of certain types of these flexible steel frames under test-load conditions, but they know a great deal more than Mr. Gregory seems to imagine. Any tower designers who work to B.S.449 stresses would soon be out of business, and they must rely on stress curves built up from test experience with various allowances for end fixing conditions, as no formulae could cover the infinite number of variables which arise.

His comments on tower testing and Load Factors are misleading, and in referring to "Load Factors" obtained under test he is putting the "cart before the horse." Transmission towers are designed for whatever Load Factors or Safety Factors are specified, the whole point being that test loads are applied which include such factors—they are working loads multiplied by the factor, and at that stage certain members would be at their yield value as ties, and very close to crippling value as struts. Beyond that point an overload of a few per cent. is all that might be expected to cause failure with a competitive design. Once the strut starts buckling the transition through yield, plastic and breaking stress is instantaneous and quite distinct from the behaviour of a tie or beam. The process during this transition period will vary with every different shape, and it is of no practical value to the designer, whose criterion must be the safe ultimate load which the strut will be able to carry without permanent deformation.

A. E. PERCIVALL,
Managing Director,
Transmission Tower Division,
Blaw Knox Limited, London.

Sixth International Congress on Large Dams

To the Editor, WATER POWER

The coverage given in the December issue of WATER POWER on the Sixth International Congress on Large Dams is quite complete and very interesting. I am sorry, therefore, that I must call to your attention your comment on the discussion which I presented at the Congress on Question 23 which appears on page 456.

I refer to the last sentence of your short comment which reads: "He concludes that fly-ash concrete should be placed 90 days before freezing starts." What I actually said was "The freezing and thawing tests indicated that when there is a deficiency of entrained air, the fly-ash concrete, if cured less than 90 days before freezing starts, will fail at a much lower number of cycles of freezing and thawing than regular concrete. It is necessary, therefore, to use an air-entraining solution in fly-ash concrete to obtain adequate durability on concrete cured less than 90 days before freezing starts."

You can easily see how the two statements are widely different, and it will be appreciated if you will report the correction.

GEO. K. LEONARD,
Chief Engineer,
Tennessee Valley Authority,
Knoxville, Tennessee.

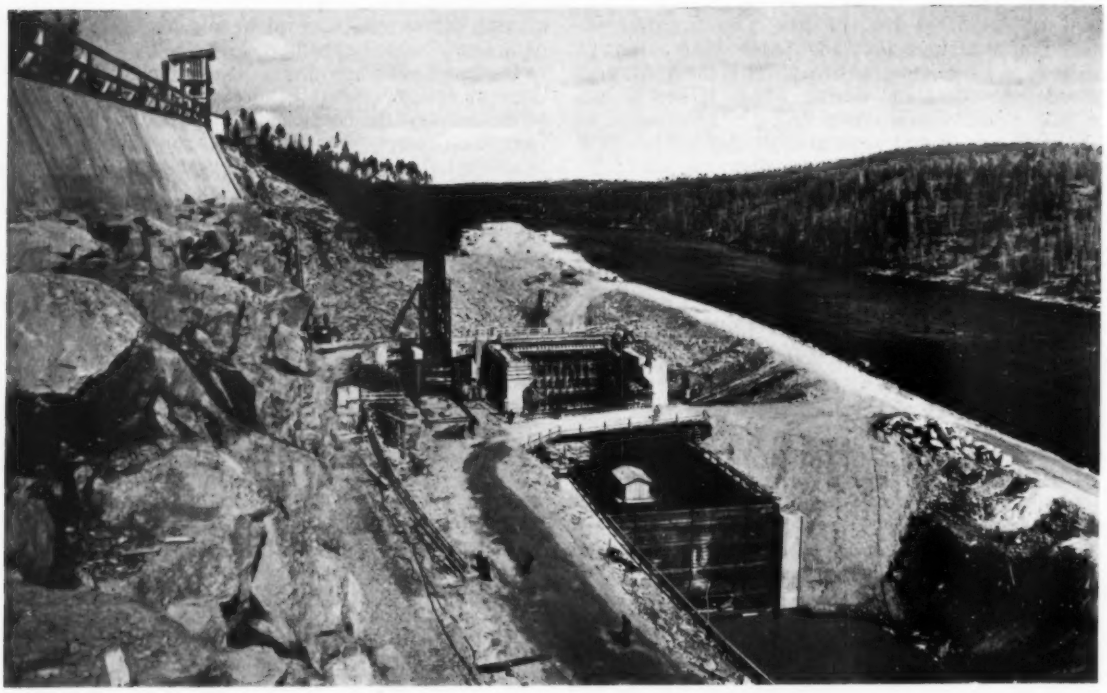

Fig. 1. No. 1 spillway of Pirttikoski dam under construction

The Pirttikoski Power Project

A brief account is given of a new station being built on the Kemi river in North Finland

By HARRI SISTONEN, Diploma Engineer

PIRTTIKOSKI is situated in North Finland some 25 km. south of Lake Kemijärvi. The name derives from rapids in the Kemi river which flows into the Gulf of Bothnia. The power potential of Pirttikoski is owned by the Kemijoki Company and the plans for the new power station have been drawn up at the headquarters of the Imatra Power Company (Imatran Voima Osakeyhtiö).

Hydrology

The catchment area above the Pirttikoski dam amounts to 27,795 sq. km., of which lakes comprise 2·5%. The measured discharge rates vary as below:—

maximum flow	—	2,150	cu. m. per sec.
mean high flow	—	1,555	,,
mean flow	—	283	,,
mean low flow	—	92	,,
minimum flow	—	65	,,

To secure a more balanced flow work has been commenced on the upper part of Kemijärvi (Lake Kemi), and plans are being made for the construction of artificial reservoirs along the upper tributaries of the river, namely, at Porttipahta, Lokka, Saija and Kuttusvaara. The combined capacity of the natural basin of Kemijärvi and these artificial reservoirs will be approximately 5,800 million cu. m.

Geology

At Pirttikoski the river flows through a fairly steep and high-banked valley, about 150 m. wide, cut out of a broken zone of bedrock, mainly of granite. In the southern part of the area there is some micaceous gneiss, into which the granite has intruded as veins and dikes.

Diamond drillings proved that the rock was fairly solid at the intake, power house and throughout the larger part of the tailrace tunnel. At the end of the tunnel and in the tailrace channel, however, the rock was of inferior quality.

Directly above the bedrock there is a layer of till varying in thickness between 1 and 60 m. Overlying this layer there are patches of medium to fine silt and of gravel covered by coarse silt and sand. Along the axis of the dam there is a layer of sand and gravel ranging from 2 to 7 m. in thickness. Below this a sandy moraine extends down to the rock. This moraine is not homogeneous, but contains well defined areas varying from sand to gravel. Towards the

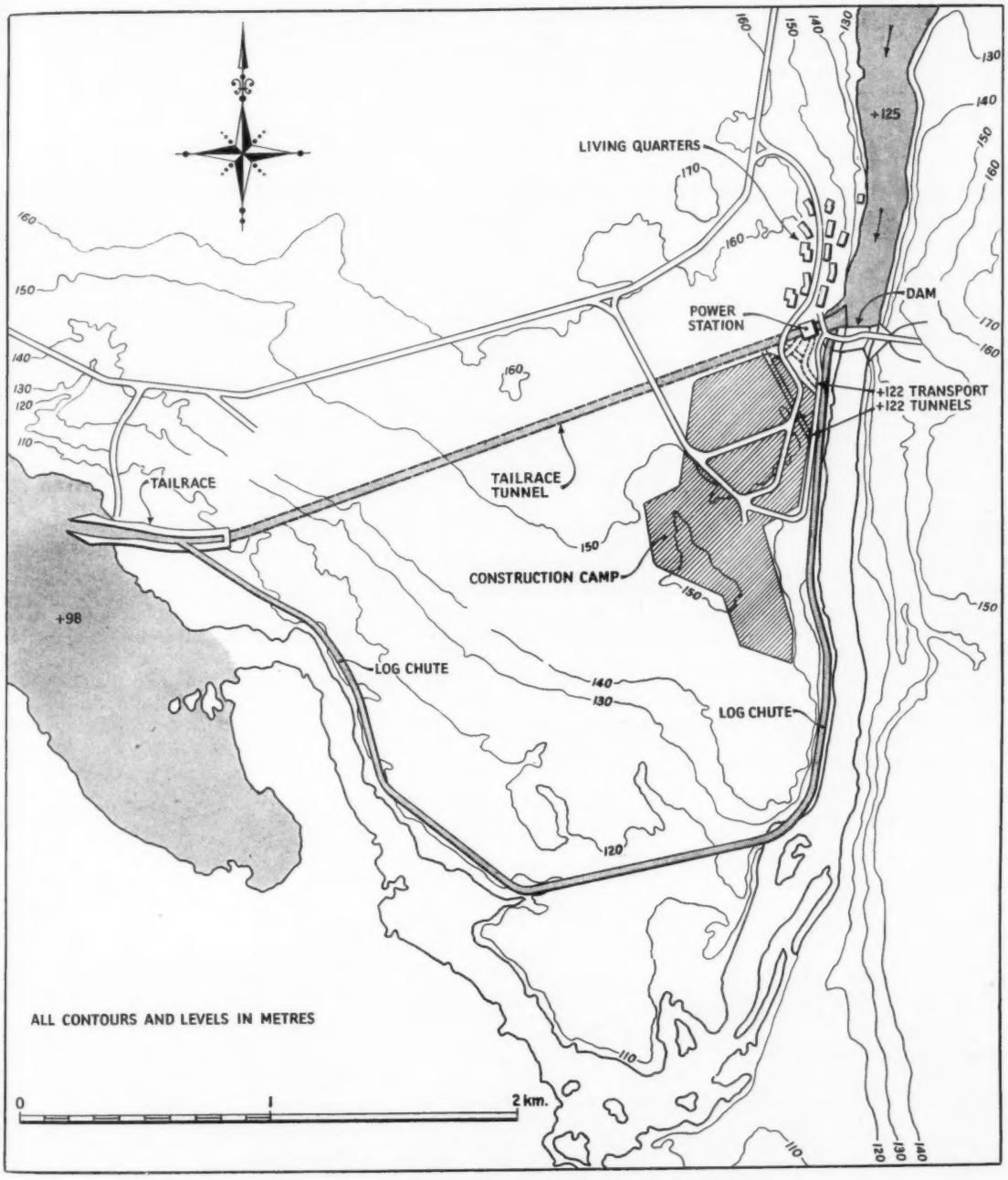

Fig. 2. General map of the Pirttikoski development

eastern shore there are silty intermediate layers.

General

The rated discharge of the power plant is 500 cu. m. per sec., divided evenly between two units. The ratio between the rated discharge and the mean flow will thus be 1·77. The headwater level is 125 m. and the tailwater level 98 m. so that the gross head is 27 m. and the estimated net head 26·50 m. After regulation the anticipated production of energy will be 550 million kWh per annum. Numerous different possibilities were considered to find the most economical solution, the most favourable being that shown in Fig. 2, in which the intakes above the dam consist of tunnels leading to the power house immediately below. The water flows through the tailrace tunnel across the Pisiö headland before it rejoins the Kemi river. The total length of the tailrace tunnel and the tailrace channel is 3 km.

Regulating Dam

The regulating dam is formed by the following

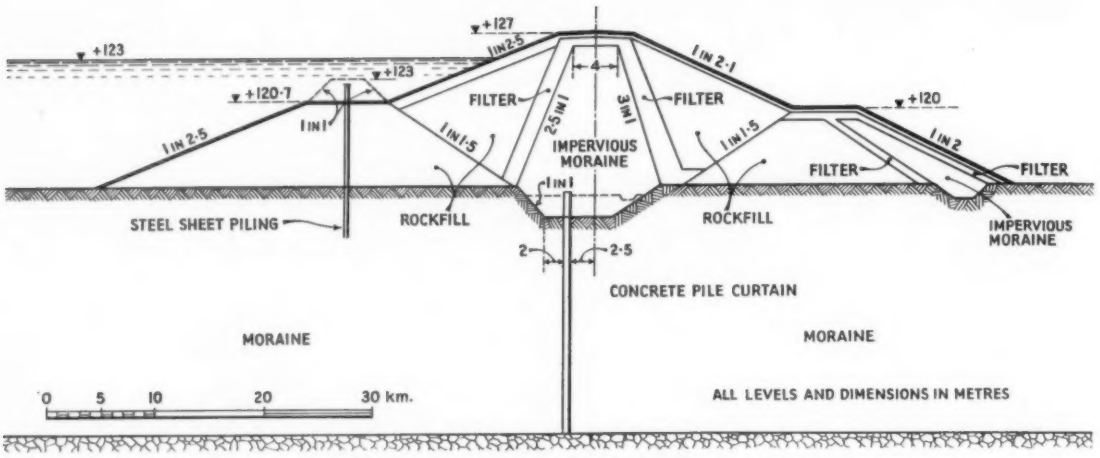

Fig. 3. Section through Pirttikoski dam

structures starting from the eastern bank: No. 2 spillway which has a sluice gate at the bottom, the earth dam, No. 1 spillway, and the intake structure of the log chute.

The No. 1 spillway, which is situated in solid rock and was designed as a result of tests carried out on a scale model, has no end sill. The characteristics are: height 12 m., width 16 m. and discharge capacity 1,300 cu. m. per sec.

The No. 2 spillway is 7·5 m. high, 16 m. wide and will be equipped with a sluice gate measuring 3·5 by 15 m. This sluice gate is divided into two halves separated by a central partition wall 1 m. thick. This spillway is provided with a stilling basin and an end sill because the water will be discharged through the spillway on to gravel instead of rock, as in the case of the No. 1 spillway. In the event of excessive flooding (+125 m.), the overflow spillway will be able to discharge 670 cu. m. per sec. and the sluice gate 370 cu. m. per sec. Both spillways will be closed by tainter gates and the sluice opening by a fixed-wheel gate. Stoplogs will be used to close the spillways when required.

The earth dam, seen in section in Fig. 3, will be protected by cofferdams during construction. As may be seen from the drawing, the earth dam will be placed on the moraine overlying the river bed. This foundation, consisting of gravel and till down to the bedrock,

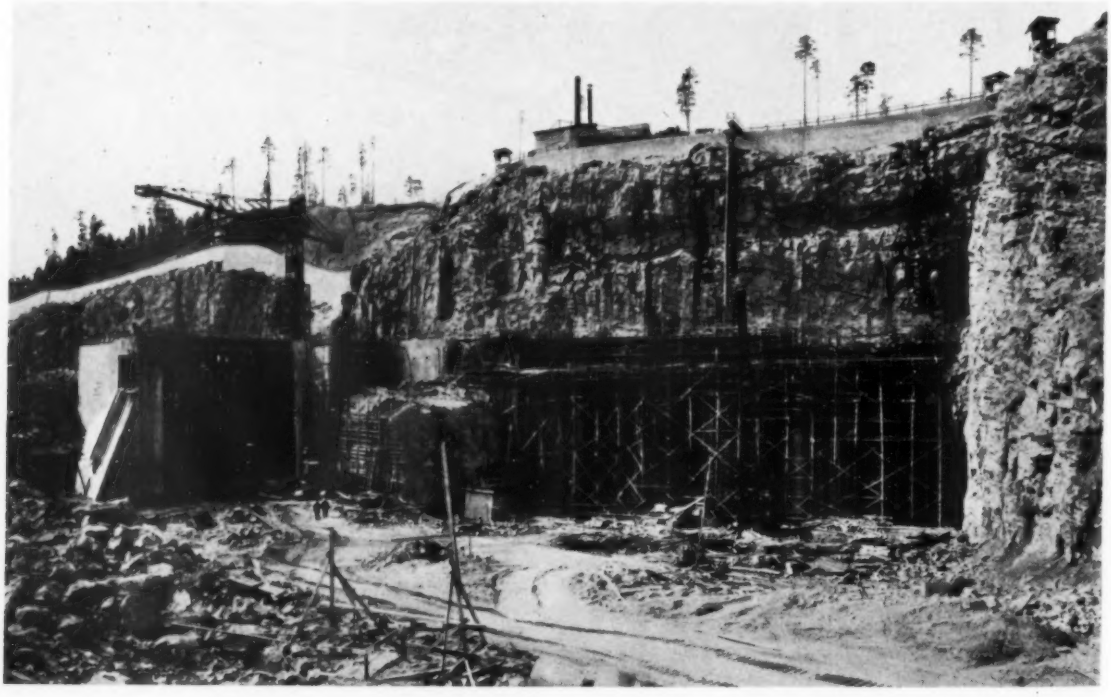

Fig. 4. The intake for the power station under construction

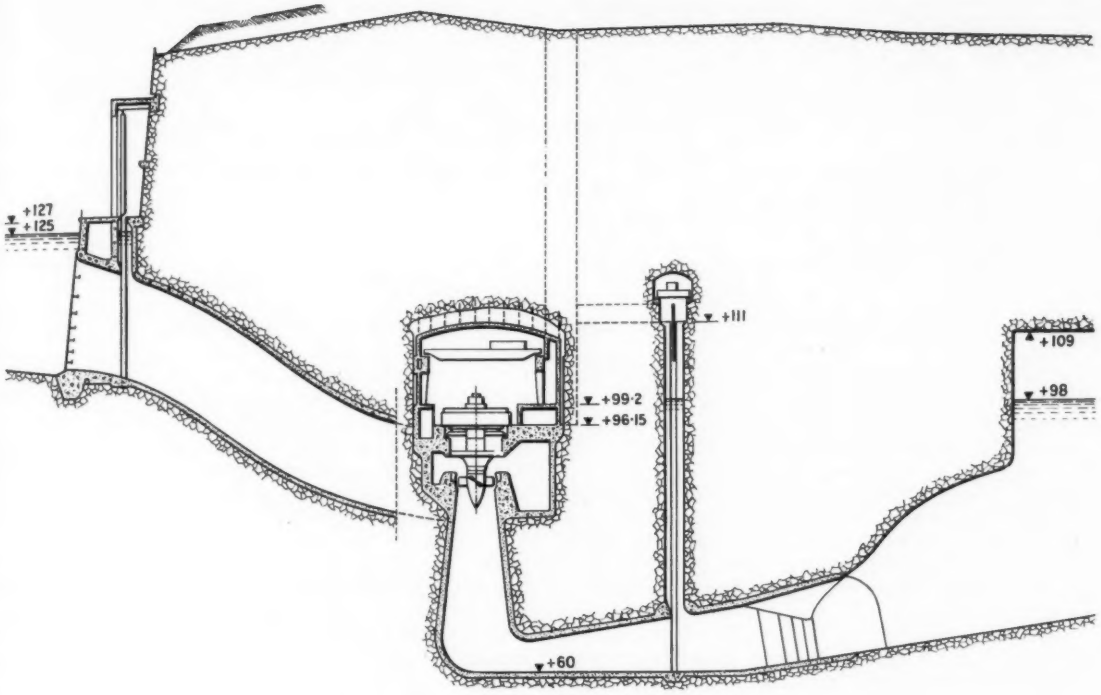

Fig. 5. Section through Pirttikoski power station

will be made impervious either by the ICOS-Veder method or by grouting, or by some other suitable process to be decided after the investigations have been completed. The earth dam will itself have a central moraine core, $k = 10^{-6}$ cm. per sec., and will be surrounded by filter zones, while the main body of the dam will be constructed of broken rock; upstream of the dam the cofferdams will be left in position. The length of the earth dam will be approximately 170 m., the width at the top 7 m. and the maximum height 18 m.

The concrete log-chute intake will have a capacity of 10,000 logs per hour. All the concrete structures, i.e. both spillways and the cut-off trench of the log chute, will be grouted to a depth of about 9 m. The log chute proper will follow the levels of the terrain and end up at the tailrace channel. Its total length will be approximately 5·5 km.

Intakes

The intake structures are situated on the western bank, next to the regulating dam with which they form an angle of 42°. The mouth of each intake is divided by two partitions, forming three equal divisions which reduce the size of the gates and lighten the roof structures. These structures will be made of concrete which will be anchored to the rock. Both the vertical and horizontal cut-off trenches will be grouted, and behind them drains will be laid to collect seepage water. The filtration plant for the cooling water of the generators will be situated on the upper part of the intake.

Only one set of the three gates to equip one intake has been ordered. They are of the vertical fixed-wheel type measuring 6·2 by 15 m. and will be operated by a gantry crane. The choice of this system was determined mainly by two factors: first, since wicket gates will be used to close the machinery in an emergency, emergency gates proper will not be necessary; second, the cost involved will be some 40% less than that of emergency gates.

The intakes, shown in Fig. 4, are 60 m. long and have a cross-sectional area of 130 sq. m. For technical reasons, blasting work will be carried out from both ends. The walls and the floor will be covered with a concrete lining about 40 cm. thick, which will be anchored to the rock; the ceiling, however, will be sprayed with concrete in two lightly reinforced layers to a thickness of from 5 to 15 cm. The reinforcement net is attached by anchor bolts, which are 22 mm. in diameter and spaced at 1 m. intervals, and are 5 m. in length in the middle of the cross-section, diminishing to 2 m. towards the sides. The thickness of the rock wall left between the intakes is 16 m.

Power House

The machine hall and the control room will be excavated from the solid rock, but the switching plant will be built on the surface. A crack runs diagonally across the roof of the entire machine hall and to make it secure roof bolting with 22 mm. bolts has been carried out across the full length of the ceiling. Special attention has been paid to reinforcing the fissured portion and it is also proposed that three arches should be cast to buttress the ceiling. The surface of the rock will be sprayed with concrete in two layers of some 3 cm. each, in which welded netting (3 mm. wire, 10 cm. mesh) attached to the anchoring bolts will be used as reinforcing material. Underneath this, a false waterproof ceiling will be built of impregnated wood, but-

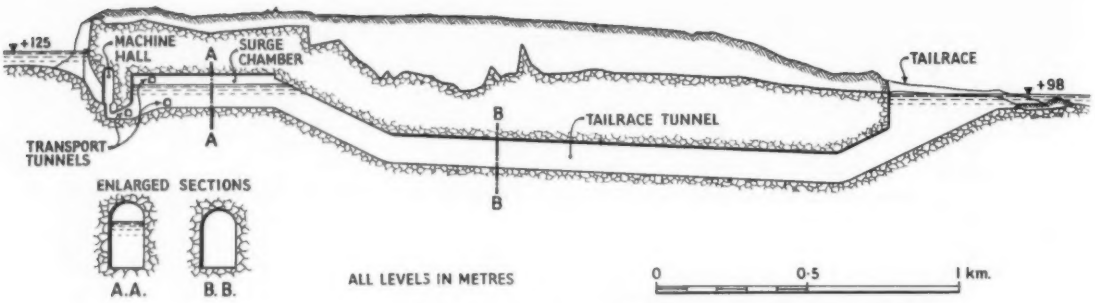

Fig. 6. Longitudinal section (vertically distorted) showing the tailrace tunnel

tressed by steel arches set up at intervals of about 3 m. The top of this wooden ceiling will be covered with bitumen felt layers to prevent water seepage.

The walls will be lined with reinforced-concrete sections set off from the rock. This will enable any seepage water between the bedrock and the concrete wall to be drained off. A system of drainage holes will also be drilled into the rock both upward and downward so that the directions of possible leakage in both the rock and later grouting can be gauged should this prove necessary.

The concrete scrollcase will be 22 m. in diameter and the chamber for it will be prepared by the so-called "smooth-blasting" method. This will enable the walls and floor of the scrollcase to be covered with a layer of reinforced concrete 20 mm. thick.

The shaft carrying the lift and busbars is situated at the southern end of the machine hall; it rises from the turbine level (+96·15) and terminates at +168·80. The dimensions after blasting are 5 m. by 5·60 m. and the shaft is lined with concrete 20 cm. thick, making final dimensions of 3·5 by 4·1 m. The shaft is 34 m. in height and was excavated by first taking out a 1·80 by 2·30 m. core. A Joy-Challenger wagon drill was used, loading and blasting being carried out from above. When this pilot shaft was completed, drilling out to the final dimensions was performed by using hand drills and normal drills in stages, proceeding from the top. The pilot shaft was completed in two weeks and the amount of explosives used was 7 kg. per cu. m. The total cost, including removal of the material, was $14 U.S. per cu. m. (solid measure).

Draft Tube

The draft tube consists of a vertical section 25 m. high and a horizontal section 50 m. long. A pilot shaft, as described in the foregoing, will be sunk for the vertical section. The upper part will have a steel-plate lining backed by reinforced concrete up to the point

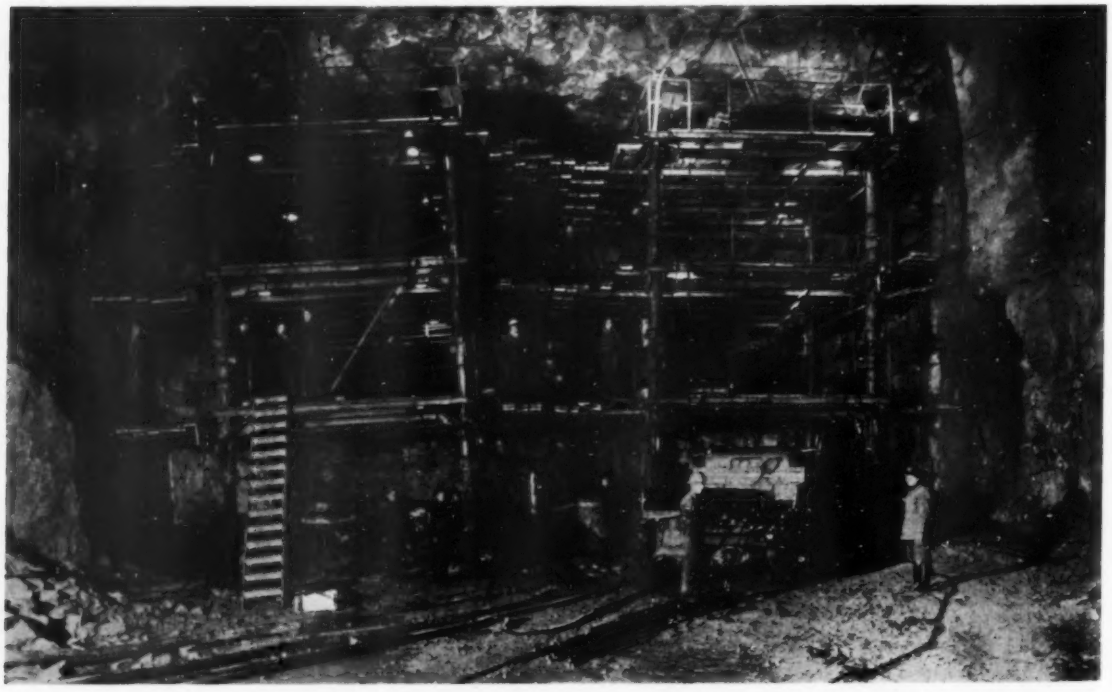

Fig. 7. Drilling jumbo in the tailrace tunnel

where the velocity of the water is about 6 m. per sec. Below this part the rock will be covered with a layer of reinforced concrete varying in thickness from 20 to 30 cm. Heading technique will be adopted for the horizontal section, the mucking being done with a Landsverk L65-B excavator. The treatment of the roof, walls and base of the horizontal section of the draft tube will correspond to that of the intake.

Surge Chamber and Tailrace Tunnel

The surge chamber, starting at +80 m. and rising to +109 m., has a width of 16 m. below the +105·00 level and then expands to a diameter of 17·5 m. It was designed on the basis of Thoma's stability criterion on the assumption that both units are simultaneously shut down when operating at a water flow of 550 cu. m. per sec. (10% overload). The tunnel is being driven with a top heading of 165 sq. m. in area and a lower section as a bench about 19 m. high.

The cross-sectional area of the tailrace tunnel is 350 sq. m. and the width is 16 m. Blasting is being carried out in two stages: the upper part as a heading of about 150 sq. m. and the lower part as a single bench of approximately 200 sq. m.

The tailrace channel in which the tunnel terminates will discharge the water into the quiet stretch of the Kemi river known as Autinjyrhämä; the channel is about 450 m. long and has an operative cross-sectional area of about 350 sq. m. It is mainly excavated from rock and has vertical walls.

The tailrace tunnel is being advanced from two faces only: at the outflow end spoil is carried through the channel itself. At the upper end, however, a pair of construction adits 9 m. wide and 6 m. high and having a combined length of about 900 m. had to be driven. The northern auxiliary tunnel branches off into three sections of which the uppermost leads into the machine hall and will be used as the transportation tunnel when the power house is completed. The second branch leads to the gate chamber of the draft tube and the third or lowermost branch leads into the upper section of the surge chamber. The last mentioned branch will operate in the completed plant both as a part of the surge chamber (the horizontal area is approximately 1,000 sq. m.) and as an air vent for the surge chamber. The southern auxiliary tunnel, again, branches into two parts, one leading into the draft tube and the other into the lower part of the surge chamber. The maximum slope in the auxiliary tunnels is 1 : 7. The entire horizontal area of the surge chamber is 8,000 sq. m.

Tunnelling Equipment

The drilling equipment used in excavating the tunnels includes: Atlas Copco BBC 21 W and Tampella T10CW drills, both of which are fitted with air legs. For the most part Sandviken Coromant $\frac{7}{8}$ in. steels have been used, ranging from 1·6 m. to 9·6 m. in length and fitted with chisel bits. The jumbo is a two-decked steel structure in the auxiliary tunnels and a three-decked in the main tunnel. Both have mechanical jacks.

A Landsverk L65-B excavator (0·9 cu. m. dipper) is used for mucking out in the auxiliary tunnels. A Bucyrus-Erie 88-B excavator (3 cu. m. dipper) is used in the main tunnel. Both models are operated by electricity.

Euclid trucks of 14 and 20 tons capacity and 30 ton Caterpillar trucks are being used for the removal of debris. A 6 in. diameter air pipe operating at a pressure of 7 kg. per sq. cm. is used in the tunnel work in conjunction with a 3 in. diameter water pipe for drilling (operating at a pressure of 5 kg. per sq. cm.), a 6 in. drain pipe and a 1·2 m. diameter ventilation tube.

TABLE I.—MATERIAL AND WORK RESULTS

Auxiliary tunnels (54 sq. m.)	
drilling	1·70 m. per cu. m.
dynamite (45%) ...	1·00–1·20 kg. per cu. m.
daily advance ...	3·50 m.
cost	$6·0 per cu. m.
loading (Landsverk)	30 cu. m. per hour
Main tunnel (150 sq. m.)	
drilling	1·30 m. per cu. m.
dynamite (45%) ...	0·70 kg. per cu. m.
daily advance ...	6·00 m.
cost	$4·0 per cu. m.
loading (B.E88B) ...	130 cu.m. per hour

Both in the main tunnel (20 drillers) and in the auxiliary tunnels (8 drillers), the loading, wiring and firing are carried out in one shift, while the mucking out is done in two shifts. In both the main and auxiliary tunnels 0·5 sec. delay-type detonators are used.

Power Plant

Each of the two vertical-shaft Kaplan turbines is designed for a rated discharge of 250 cu. m. per sec., a net head of 26·5 m. and a full-load capacity of 60,000 kW. They have a runner diameter of 5·80 m. and the speed is 115·4 r.p.m. They were manufactured by Escher Wyss A.G.

The alternators, supplied by Siemens-Schuckertwerke A.G., are both designed for a capacity of 70,000 kVA, 0·95 power factor and 13·8 kV. There are four single-phase transformers, each 155/105/25/25 MVA and two 45 MVAr, 14·8 kV reactors. The switchyard is equipped with a 400 kV and a 200 kV unit.

The machine hall is equipped with two travelling cranes with hooks to lift 150, 30 and 10 tons. The corresponding lifting heights are 17, 17 and 50 m. and the lifting speeds 0·6, 3 and 6 m. per min. In addition, in the transformer room there is a 125/10 ton travelling crane, having a lifting height of 15·7 m. and lifting speed 0·5 m. per min.

Permanent and Temporary Buildings

Pirttikoski will serve as a permanent base for the upper Kemijoki river project. Among buildings and structures put up during the work period there are barracks for some 800 men, a repair shop and storage area and a concrete plant with an output of approximately 300 cu. m. per day.

There is also permanent accommodation for 51 families, a permanent repair shop, a building for the switching plant, an assembly tower for the transformers, a recreation hall, clubhouse, and a servicing building which includes a central heating plant, sauna baths, etc.

Costs and Work Time

It is estimated that the total cost will be $27·5 million (U.S.) which, however, does not include the Pirttikoski share of regulating the upper waterways. The work will be completed during 1959.

Rupture Studies on Arch Dams by Means of Models

A method of determining the coefficient of safety of arch dams by means of rupture tests on models has been devised at the National Civil Engineering Laboratory, Lisbon

By MANUEL ROCHA* and J. LAGINHA SERAFIM†

PART ONE

THE main objective in the analysis (both mathematical and experimental) of an arch dam is to determine its coefficient of safety, i.e. to find out how far the structure is from rupture under the possible maximum loads. If all the points of a dam are approximately at the same " distance " from rupture, it can be said that the dam is well designed, and in such a case the closer that " distance " the more economical is the dam. Structures must be designed to approach rupture, although rupture itself must be avoided. The means that have been used, so far, for analysing arch dams are based on the determination of the stresses by assuming an elastic behaviour of the material of the dam and of the foundation. From the comparison of such stresses with the ultimate strength of the material, determined in small samples by compression tests, the safety of the structure is estimated and judged.

The use of models, allowing a reliable determination of the elastic stresses, has made it possible to improve the design of arch dams by giving them suitable shapes and very economical thicknesses. However, the use of models also gives the possibility of a direct estimation of the safety of a dam with respect to rupture. This can be achieved either by increasing some of the loads acting on the models up to rupture or by decreasing the strength or the thickness of the materials used, or even by decreasing the quality of the foundations, until the normal loads produce failure.

Stresses arising from temperature,[1] pore pressure or shrinkage of the concrete have probably to be neglected until they can be conveniently reproduced in an advanced stage of model testing up to rupture. The results already obtained in rupture tests, although at a preliminary stage, were considered of sufficient interest to be presented here. But a much wider research programme, using various materials and techniques, is being undertaken at the Laboratório Nacional de Engenharia Civil, Lisbon.

Safety of a Dam

A structure can undergo rupture due to one or more of the following causes: The design loads being exceeded; the computation methods being deficient and leading to an overestimation of the strength of the structure; the construction being deficient and the materials of the structure or of its foundation not having the properties assumed in the design. In the case of an arch dam, the loads cannot increase beyond their maximum design values—the dead load is constant, temperature and uplift have maximum possible values, and the live load—the hydrostatic pressure—cannot rise above a certain value unless the spillway becomes deficient. But even in this case, for a high dam, the increase of the hydrostatic pressure when the crest is overflowed is not very important and can be taken into consideration in model tests if desired. What seems to be more important for rupture, in the event of overflow, is the destruction of portions of the dam or of the foundation by the water falling from the crest.

As to calculation methods, although they are subject to severe criticism, the usual safety factor in arch dams is sufficient to cover any faults in the methods adopted. This is corroborated by the fact that accidents or even serious deficiencies of behaviour as the result of inadequate calculation are unknown. The only trouble found, and this frequently, are cracks resulting from tensile stresses, which at times require repair work, but without any risk of safety as long as such work is undertaken in good time. Even if improved knowledge leads to the introduction of arch dams with appreciably different characteristics from those existing at present, the fact of having models by which stresses can be determined very accurately makes the possibility of rupture through faulty calculation very unlikely.

Deficient construction, lack of strength in the concrete or foundations should receive more attention than other causes, although no complete failure of a dam due to concrete rupture is known. The only cases to be noted are some serious situations which have arisen from the chemical alteration of the concrete, especially alkali-aggregate reaction. But if it is desired to make the maximum use of the strength of the concrete and foundation materials it is imperative to foresee the behaviour of a dam in the event of any progressive weakening, and to determine the strength of the concrete at which rupture will occur.

Thus a real conception of "safety factor" can be defined as the ratio between the compressive strength of the concrete to be used in the arch dam and the compressive strength of a concrete for which the rupture of the dam would occur under the maximum design loads acting on the dam. A similar definition can be given for the strength of the foundations.

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It is evident that if the materials were elastic up to rupture and if a redistribution of stresses due to cracks produced by tensile stresses did not occur, such a factor would be equal to the ratio between the ultimate strength of the concrete to be used in the dam and the maximum compressive stress produced by the maximum design loads. This is based on the assumption that the maximum compressive stress at a point is what determines rupture, independently of the value of the other principal stresses. This is not far from the truth, once the maximum compressions develop at the surface where the state of stress is biaxial (downstream face) or with a third stress of reduced value (hydrostatic pressure at the upstream face).

The difference between these two ratios results from the redistribution of the stresses along the dam due to the inelasticity of the concrete in the neighbourhood of rupture. Hence the consideration of a " factor of adaptation " becomes necessary as the ratio between the maximum compressive stresses that would exist in the structure, in the elastic range, for the maximum design loads and the compressive strength of the material for which the rupture of the dam would take place under those loads.

The main adaptation phenomena in the rupture of a dam are probably the redistribution of stresses across the thickness, and transfer of compressions from the most stressed regions to the neighbouring regions. There will also be some other redistributions across the thickness due to the skin effects produced by temperature changes.

For a structure on which the loads cannot increase beyond certain limits, as is the case of an arch dam, the only satisfactory definition of a safety factor is the one already given. In view of the complexity of the phenomena in question it is not possible to determine this factor except by means of models. The next section will give details of the conditions which the models will have to satisfy.

One important consideration in the safety of an arch dam is the existence of regions in the dam where the strength of concrete is appreciably lower than average, or the existence of weaker spots in the foundation. Such a condition can have a definite influence on the failure of the structure and should be analysed in detail in each particular case.

No doubt, when designing an arch dam, the parameters about which the greatest uncertainty exists are usually those which concern the foundation. On occasions the foundation properties are not well known, either because of insufficient exploration or due to the occurrence of heterogeneous features difficult to discover by prospecting. At other times the properties may alter through exposure to air or immersion in the reservoir water and its percolation. Also the foundation may be weakened by downstream erosion, especially that induced by overflow of the dam. The accidents and deficient behaviour found in arch dams so far are almost exclusively due to the foundations.

In situ deformability[2] and shearing tests[3] and laboratory investigations on the properties of the foundations improve our knowledge of their behaviour, but many unknown factors still remain. Rupture tests on models, in which weaknesses in the foundations are reproduced or in which the whole foundation is reproduced by very weak materials, can allow certain factors of safety to be estimated.

Important rupture tests on models of arch dams have been carried out in Italy.[4,5,6] In these the intensity of the hydrostatic pressure for the maximum water level was increased by proportionally increasing the forces applied at the upstream face by means of jacks, but the forces reproducing the weight, which were applied by means of wires inside the models, remained constant. This does not correspond to a physical condition in the prototype. Such tests have shown a large safety margin to exist even in very thin arch dams, but the lines of rupture and their position probably will not correspond to their position in a dam. The first rupture always occurs by tension, in the regions of maximum tensile stresses when the hydrostatic pressure only is increased.

Rupture tests considered to have more significance are those in which a safety factor is sought in relation to the weakness of the concrete or of the foundations or of parts of them.

General Mechanical Similarity Conditions

Let us consider the model of a structure to a scale of $1/\lambda$, and assume that the homologous displacements, deformations and stresses are proportional up to rupture. The following are the conditions which the material of the model should satisfy and the loadings which should be applied to it[8]:

As to the material, for proportionality to be obtained, it will have to have a stress σ strain ϵ diagram obtained from that of the material of the prototype multiplying the ordinates and abscissae by constants $1/\alpha$ and $1/\beta$ (Fig. 1). For homologous points m and p of the diagrams there will be the relations:

$$\sigma_m = \frac{1}{\alpha}\sigma_p \quad \text{and} \quad \epsilon_m = \frac{1}{\beta}\epsilon_p$$

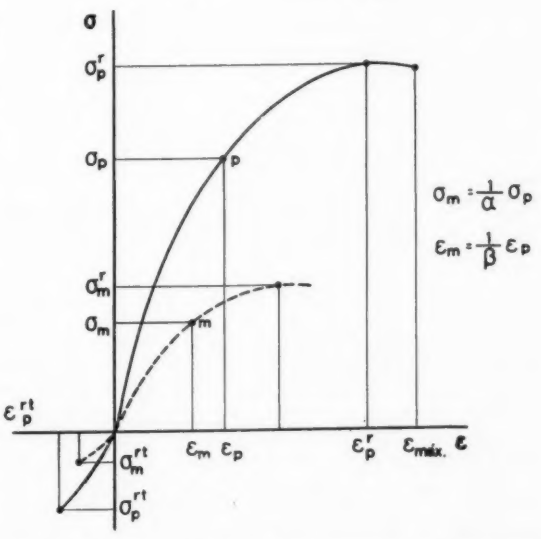

Fig. 1. Stress-strain curves for concrete and for the material of the model

This condition is necessary but not sufficient. The general condition to be satisfied by the material of the model is similar but must apply for a triaxial stress. This general condition is the following: let ϵ_p be the strains produced in a parallelepiped of the material of the prototype submitted to a triaxial state of stress

by the application of the stresses σ_p at its faces; the material of the model has to be such that a parallelepiped submitted to stresses of homologous intensity $\sigma_m = (1/\alpha)\sigma_p$ will undergo homologous strains $\epsilon_m = (1/\beta)\epsilon_p$.

The verification of this general condition would be very laborious, but in practice, as long as the materials of the prototype and the model are of an analogous nature, it will be sufficient to verify the existence of the condition laid down for tensile and compression tests (Fig. 1). It should also be noted that, given the diagrams of two materials, it is usually possible to determine the scales $1/\alpha$ and $1/\beta$ once the shape of the curves is analogous. The forces to be applied to the model should be to the scale of $1/\lambda^2\alpha$, that is, if the forces applied to the prototype are F_p, the homologous forces to be applied to the model should be $F_m = (1/\lambda^2\alpha)F_p$.

Once the materials and loadings satisfy the above conditions the homologous displacements, δ_m and δ_p, strains, ϵ_m and ϵ_p, and stresses, σ_m and σ_p, are proportional:

$$\delta_m = \frac{1}{\lambda\beta}\delta_p$$

$$\epsilon_m = \frac{1}{\beta}\epsilon_p$$

$$\sigma_m = \frac{1}{\alpha}\sigma_p$$

In the event of the deformations being influenced by time, the conditions enunciated should be verified for homologous times θ_m and θ_p, that is, a time scale $1/t = \theta_m/\theta_p$ should be taken into consideration and the conditions previously laid down should be verified for times at this scale.

It must be noted that similitude calls for $1/\beta = 1$. In practice, however, values different from unity can be adopted when the influence of displacements on the state of stress is not relevant.

In the case of a dam, let

γ_m, γ_p be the specific weight of the liquids which produce the hydrostatic pressure in the model and in the prototype,

γ'_m, γ'_p be the specific weights of the materials of the model and of the prototype, then:

$$\frac{1}{\rho} = \frac{\gamma_m}{\gamma_p} \text{ and } \frac{1}{\rho'} = \frac{\gamma'_m}{\gamma'_p}$$

The forces, that is the weight and hydrostatic pressure, should be to the scale $1/\lambda^2\alpha$. Hence, for the hydrostatic pressure $1/\lambda^3\rho = 1/\lambda^2\alpha$ and for the weight $1/\lambda^3\rho' = 1/\lambda^2\alpha$. That is $1/\alpha = 1/\lambda\rho = 1/\lambda\rho'$, which gives $1/\rho = 1/\rho'$. This means that the scale of the specific weight of the liquids which produce the hydrostatic pressure has to be equal to the scale of the specific weight of the materials of the model and the prototype.

If the compressive strengths are $\sigma_m{}^r$ and $\sigma_p{}^r$ for the materials of the model and prototype, and the tensile strengths $\sigma_m{}^{rt}$ and $\sigma_p{}^{rt}$ (Fig. 1), then

$$\sigma_m{}^r = \frac{1}{\alpha}\sigma_p{}^r = \frac{1}{\lambda\rho}\sigma_p{}^r$$

and

$$\sigma_m{}^{rt} = \frac{1}{\lambda\rho}\sigma_p{}^{rt}$$

In brief, in the model test of a dam where rupture occurs, the following conditions must hold:

$$\frac{1}{\rho} = \frac{1}{\rho'} \text{ and } \frac{1}{\alpha} = \frac{1}{\lambda\rho}$$

and the model has to be made of a material which satisfies the conditions shown in Fig. 1, especially in regard to its compressive and tensile strengths having to be at a scale of $1/\lambda\rho$.

Experimental Determination of the Factors of Safety and Adaptation

Let us see what conditions the moduli for the determination of the safety factor have to satisfy. Let us consider in the first place the factor of safety in relation to rupture due to very poor-quality concrete.

Following directly from the definition cited, in order to obtain the value of the factor of safety, successive models would have to be tested, the properties of which would satisfy the similarity conditions corresponding to concretes which are weakening progressively till rupture takes place. To each model forces corresponding to $1/\rho = 1/\rho' = \lambda/\alpha$ should be applied, the value of $1/\alpha$ being given by the material used in the construction of the model. The foundation properties would have to satisfy the similarity conditions relative to the properties of the actual rock. Such a process, however, would be long and expensive.

A technique was therefore worked out by which the factor of safety is determined by the application of successively increasing loads to the same model till rupture occurs. Let us see the conditions the model should satisfy and the loads which should be applied.

As rupture of the prototype—designed with the necessary safety—could only occur if the concrete were of a very bad quality, the material used to reproduce the concrete of the dam must also reproduce the qualities of such a concrete. The foundation must be reproduced by a material with properties similar to those of the actual rock, the scales being the same as those corresponding to the dam.

All the loadings that will be operative at rupture should be reproduced. In the tests being conducted, however, only the hydrostatic pressure and the weight of the dam are being considered. The condition $1/\rho = 1/\rho'$ demands that the forces reproducing the hydrostatic pressure and the weight of the dam should increase in such a way that a constant relation is maintained between their values.

Let $1/\rho$ be the value of the scale of specific weights for which rupture of the model took place. The scale of stresses is $1/\alpha = 1/\lambda\rho$ and therefore the model faithfully reproduced the rupture of a dam under the following conditions:

(1) that it is made of a material the stress-strain diagram of which is obtained from that of the material of the model by multiplying the ordinates by $\alpha = \lambda\rho$ and the abscissae by any value β, but with the condition that it is not far from unity,

(2) that the hydrostatic pressure and the weight be applied simultaneously and after the construction of the structure.

As to the first condition, it may happen that the concrete with a strength $\sigma_p{}^r = \lambda\rho\sigma_m{}^r$ will have a diagram with a shape considerably different from that of the material of the model. This indicates that, when choosing the material for the model, a wrong safety factor was assumed, in which case it would be necessary to test another model consisting of a material calculated to reproduce a curve showing the concrete with which

rupture would occur. This should not happen, however, once a certain experience has been gained.

Analogous consideration should be given to the foundation material, for its choice implies an *a priori* adoption of a certain safety factor. In view of the latitude with which the properties of the foundation material of a dam can be defined, however, only in cases where the factor of safety is very different from that assumed would it be necessary to build another model.

The application of weight to a finished, monolithic dam model should impose lower compressions than those developed in reality. As the function of such compressions is to compensate for the tensile stresses due to hydrostatic pressure, it is to be expected that the model tests would yield lower values for the safety factor.

When a rupture has been produced, the safety factor is:

$$\phi = \frac{\sigma_R}{\sigma_p{}^r} = \frac{1}{\lambda\rho}\,\frac{\sigma_R}{\sigma_m{}^r}$$

where σ_R is the compressive strength of the concrete to be used in the dam. The factor of adaptation has the value

$$\psi = \frac{\sigma_{max}}{\sigma_p{}^r} = \frac{1}{\lambda\rho}\,\frac{\sigma_{max}}{\sigma_m{}^r} = \frac{\sigma_{max}}{\sigma_R}\,.\,\phi$$

where σ_{max} is the maximum compressive stress that occurs in the dam under the maximum design loads.

Once the design of a dam, based on rupture tests, is fixed, the nature of the foundations may also require the safety factor to be determined for the gradual weakening of the foundation, generally at a particular spot. As the probability of rupture in a dam due both to quality of the concrete and to failure of the foundation is very remote, the dam will be assumed to consist of the concrete used in its construction.

In order to determine the factor of safety by means of tests in which the forces are progressively increased, the models should comprise:

(1) the dam, made of material similar to the concrete to be used; once this material is chosen the scales $1/\alpha$ and $1/\beta$ are fixed;

(2) places in the foundation where rupture is not foreseen should be made of materials similar to those of the site and to the same scales $1/\alpha$ and $1/\beta$;

(3) regions for which the safety factor is required should be made of materials similar to those which can be expected to cause the rupture of the dam, thus implying the *a priori* adoption of a value for the factor of safety.

Having carried out the test up to rupture by the application of forces satisfying the relation $1/\rho = 1/\rho'$, let $1/\alpha = 1/\lambda\rho$ be the value of the scale of the stresses corresponding to rupture. If the corresponding values of the compressive and tensile strengths of the concrete $\sigma_p{}^r = \lambda\rho\sigma_m{}^r$ and $\sigma_p{}^{rt} = \lambda\rho\sigma_m{}^{rt}$ be appreciably different from those which the concrete would actually have in the dam, another model will have to be built, in which places in the foundation for which a safety factor has to be determined will have to be formed of stronger or weaker materials than before, according to the previous result.

Once a satisfactory test has been performed, the safety factor for the foundation is given by the expression:

$$\phi = \frac{\sigma_{R_f}}{\sigma_{p_f}{}^r} = \frac{\sigma_{R_f}}{\lambda\rho\sigma_{m_f}{}^r}$$

where σ_{R_f} and $\sigma_{m_f}{}^r$ represent, respectively, the expected strength of the material in the regions of the foundation in question and the strength of the material used in the model which reproduced these regions.

(*To be continued*)

E.D.F. Transmission Lines

Doulton Industrial Porcelains Ltd. presented three very interesting films to a competent and critical audience at the Doulton Memorial Hall, Lambeth High Street, London, S.E.1, on January 29 and 30, 1959.

The films were loaned by Electricité de France, who were represented on this occasion by two French engineers, MM. Lafont and Mercier. The main subjects were the conversion of the Genissiat-Plessis-Gassot transmission line from 225 kV to 380 kV and the use of helicopters for overhead line inspection. The Genissiat-Plessis-Gassot line was constructed in accordance with the standard practice in 1953 involving the use of 225 kV lines designed for subsequent conversion to 380 kV. In this particular case, the line consisted of six 411 sq. mm. A.C.S.R. conductors forming two circuits which could be paired to a single double-bundled 380 kV circuit.

V-strings, joined at their lower ends, were attached to the 225 kV connections. There are disadvantages in this somewhat unusual arrangement because certain forces are produced which would not be present if the two strings were vertical.

The lower yoke for the string had to be designed in such a way that it would remain stationary even in strong winds. This was achieved by making the arms of the V intersect at the middle of two clamps, so that the stresses applied to the latter were always transmitted through the apex of the V.

Two tests were carried out on the V-string and the first of these showed that, if one arm of the V should rupture, this neither caused the other to break nor resulted in the conductors clashing. The second proved that a power arc over the insulators passed very quickly to one of the points on the arcing ring, and to the upper arcing horn, or to the earth rod fitted to one angle of the tower.

Helicopters have been tried by E.D.F. for the inspection of some 25,000 km. of 225 kV and 60 kV line for which it is responsible. The machine shown in this case was a Bell 47 D1 powered by a 200 h.p. engine with a cruising speed of 130 k.p.h.; line inspection was carried out at 50 k.p.h. Apart from the obvious advantage of speed, the helicopter can patrol normally inaccessible sections, such as valley and river spans, and it can be used for close examination of the upper parts of towers which are invisible from the ground.

The films shown were in no way connected with Doulton Industrial Porcelains Ltd., but this company specialises in the manufacture of porcelain insulators and the arrangements for this presentation were made by the company to give those interested an opportunity of seeing and hearing about an enterprising achievement in extra-high-voltage transmission.

Characteristics of Francis Turbines

The author presents the results of a statistical study of the characteristics of over 350 Francis turbine installations to enable designers to form a reliable estimate of current technological trends. It is hoped later to present further studies covering other types of turbines and electric generators*

By Prof.Ing. DINO ZANOBETTI

DESIGNERS of hydro-electric power stations know that comparisons with stations already built are of great help both in the preliminary and in the more detailed stages of design. During the stage of preliminary design a collection of statistical data is more representative of actual technology and of manufacturing trends than the formulae and curves to be found in textbooks; during the stage of detail design the study of a plant possessing similar characteristics to the one to be built may help either by confirming the solutions already found or by suggesting new solutions even outside the usual schemes.

It is these considerations which have led the author to undertake a statistical research into the characteristics of hydro-electric generating sets, and the results, critically assessed and classified, are presented here. Considerations of space have led to a subdivision of the work into a series of articles, the present one dealing only with Francis turbines. It is hoped to deal with other types of turbines and electric generators in further articles.

Scope of the Research, Limits and Methods of the present Study

In the course of the research many hundreds of plants have been reviewed, only about 350 of which have been used in the present study. The basis of selection has been the availability of data of adequate reliability, and in addition both the age and the output of the machines have been taken into account. As a rule, no consideration has been given to older plants, though some exceptions have been made for those of larger output and also for some of the best known, as they could be useful for comparison. Also, as a rule, no consideration has been given to turbines with outputs smaller than 10 MW, though, again, exceptions have been made for some very well-known plants when the full availability of data have made them useful for our purpose.

It may be seen that technological progress and variations in manufacturing trends are not changing so rapidly as to invalidate the conclusions for a few years. It is proposed, nonetheless, to bring the work up to date, as necessary, in the future.

In the method adopted well-defined curves or formulae are not given nor is it proposed to confirm those already known, though extreme and average values can readily be deduced. The intention is mainly to help designers in their search for more information concerning the plants the dimensions or service conditions of which are nearest to the case actually to be designed. As a consequence the diagrams and the tables cannot be considered in isolation from the bibliography. Many unusual solutions, in fact, are readily explained as a result of numerous considerations which cannot be treated adequately in a single paper.

Previous Work

Previous studies on the same subject—even those with different purposes in mind and based on a different approach—have been carefully taken into account. That by W. L. Voorduin[1] has been the most useful; it considers relatively few plants, mostly of the Tennessee Valley Authority scheme, so chosen as to be representative of the conditions applicable in the Authority and of the trends of American manufacturers. As such, the results differ somewhat from European practice.

Of the many other works we only mention the most recent. The paper by R. Piguet,[2] instead of relating all the dimensions to specific speed and discharge diameter, takes as a parameter the ratio of discharge flow to actual angular speed. The merit of this procedure should lie in the possibility of representing most relations as straight lines on logarithmic scales. Upon accurate examination however such representations do not appear to be very consistent with practice. Other recent works are those by P. Siltamaa[3] and by P. C. Nag and K. Madhavan[4] which supply nomograms and formulae useful in restricted ranges of application. Besides these we have made use of the extensive material widely distributed in various papers, in handbooks and in manufacturers' catalogues. Mention should be made of the reports by A. Fishmen,[5] E. B. Strowger[6] and G. Ucelli,[7] and well-known handbooks by W. P. Creager and J. D. Justin[8] and by A. E. Knowlton.[9]

Discussion of the Sources of Information

When this study was taken in hand it was intended to make use of published information only; quotation of the sources would have made reference very easy, and since every plant of importance has been described at length in the technical journals, the task seemed to be an easy one. It was soon realised, however, that this procedure was impracticable. Turbine outputs given in papers are sometimes the maximum, sometimes the "rated" and sometimes the generator outputs. The best-efficiency outputs, which are so im-

* This article originally appeared in full in *L'Energia Elettrica*, Vol. XXXIV, No. 5, 1957, and with the permission of the editor an English translation was offered to WATER POWER by the author. The tables and bibliography referred to in the text were unfortunately too voluminous to admit of publication in this journal, but they can be consulted in the original Italian publication. The conclusions to be drawn from these tables, however, are implicit in the charts which we have reproduced; the numbered points in the charts have been plotted from the appropriate entries in the corresponding tables. The good offices of the British Hydromechanics Research Association in connection with this article are gratefully acknowledged.

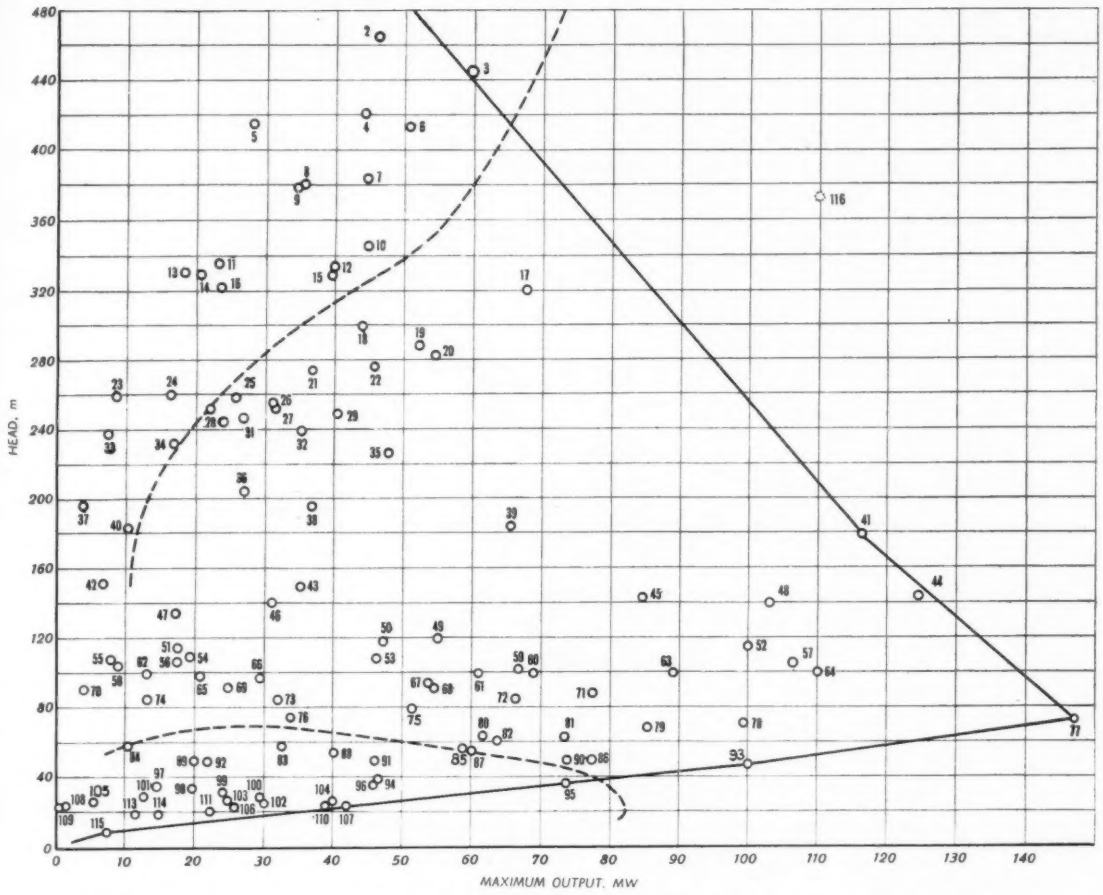

Fig. 1. Manufacturing limits of Francis turbines (1957). (Dotted lines represent the limits for Pelton wheels and for Kaplan turbines)

portant for the determination of the characteristic parameters, can only be found in a few instances when the efficiency curves are published; heads are in some cases maximum, in others average.

A critical review of the sources has consequently necessitated much work. In many cases written correspondence was necessary with the users, where they proved to be more ready to give information than the manufacturers. As a whole this work engaged the author from 1949 to 1957.

Although every care has been taken to avoid errors, no claims as to the exactitude of every figure can be made, and we shall be glad to receive corrections of errors or misinterpretations.

In the bibliography of the plants those works have been chosen which are readily available and which supply an adequate amount of information; omissions may certainly have been made and we apologise for any such.

Generally, every effort has been made to ensure that all the references quoted have been accurately examined, compared and criticised with the purpose of preparing a work which should be the most complete to our knowledge.

Of course this would not have been possible but for the help of many who collaborated by abstracting, classifying and checking the figures. The author wishes to acknowledge the contribution of his former students, now graduate engineers, E. Ciaccheri and G. Nironi who assisted at different periods from 1949 to 1955. He should also acknowledge the authoritative advice of Dr. Ing. A. Cita. Mr. D. F. Oakeshott has kindly agreed to revise and correct the English text.

Note on Specific-Speed Definition

Throughout this study use has been made of specific speed referred to metric and horsepower units. Anglo-Saxon designers still use the foot instead of the metre whereas Scandinavians have a preference for the metre and the kilowatt.

Obviously:—

n_s (m., h.p.) = 4·4 n_s (ft., h.p.) = 1·17 n_s (m., kW)

FUNDAMENTAL WORKING CHARACTERISTICS

Ranges of Use: Heads and Outputs

Fig. 1 (derived from Table 1),† shows for a number of known plants the maximum output and the rated head, its purpose being to fix the limit reached by

† *See* footnote on page 96.

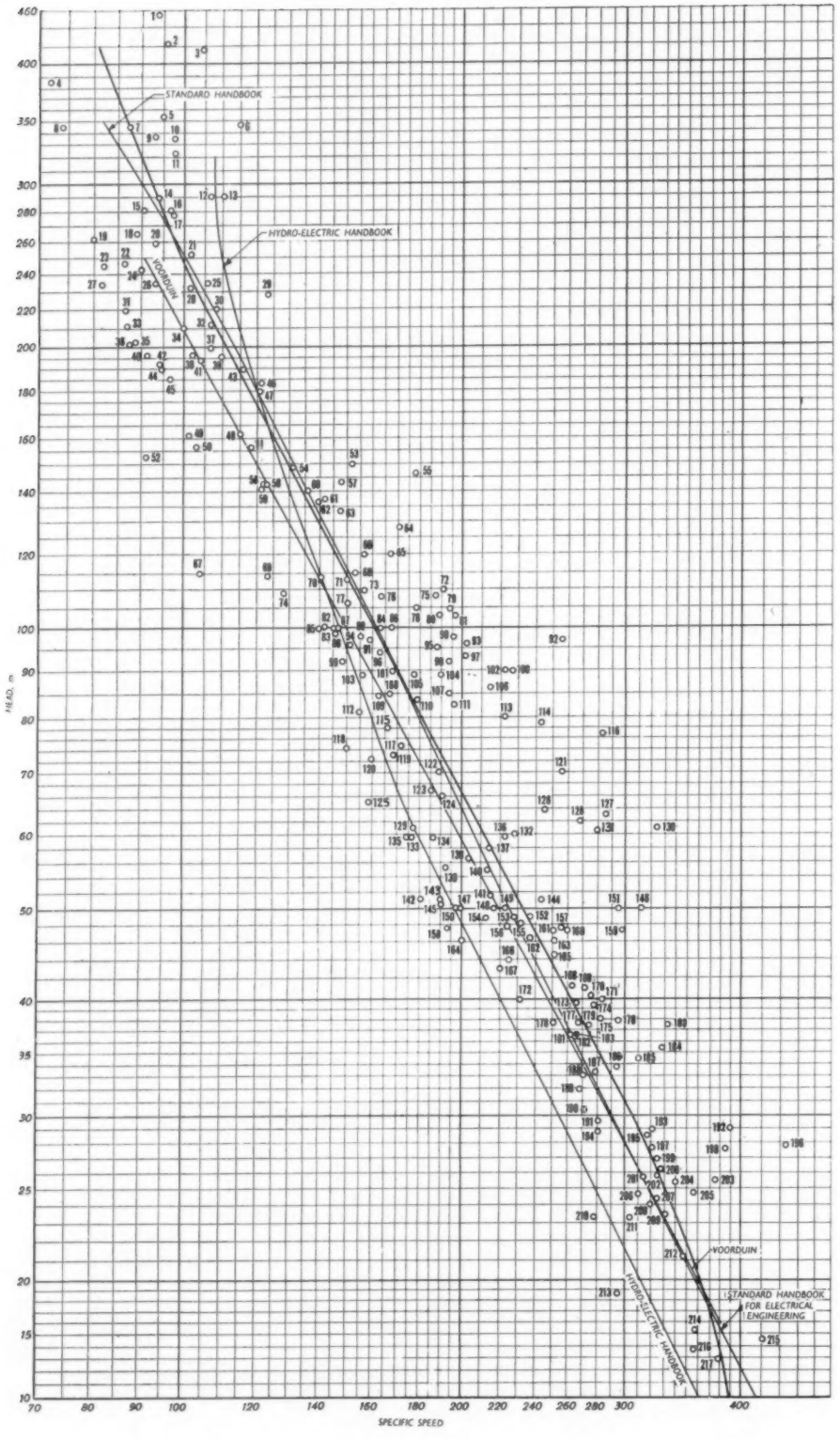

Fig. 2. Specific speed as a function of rated head

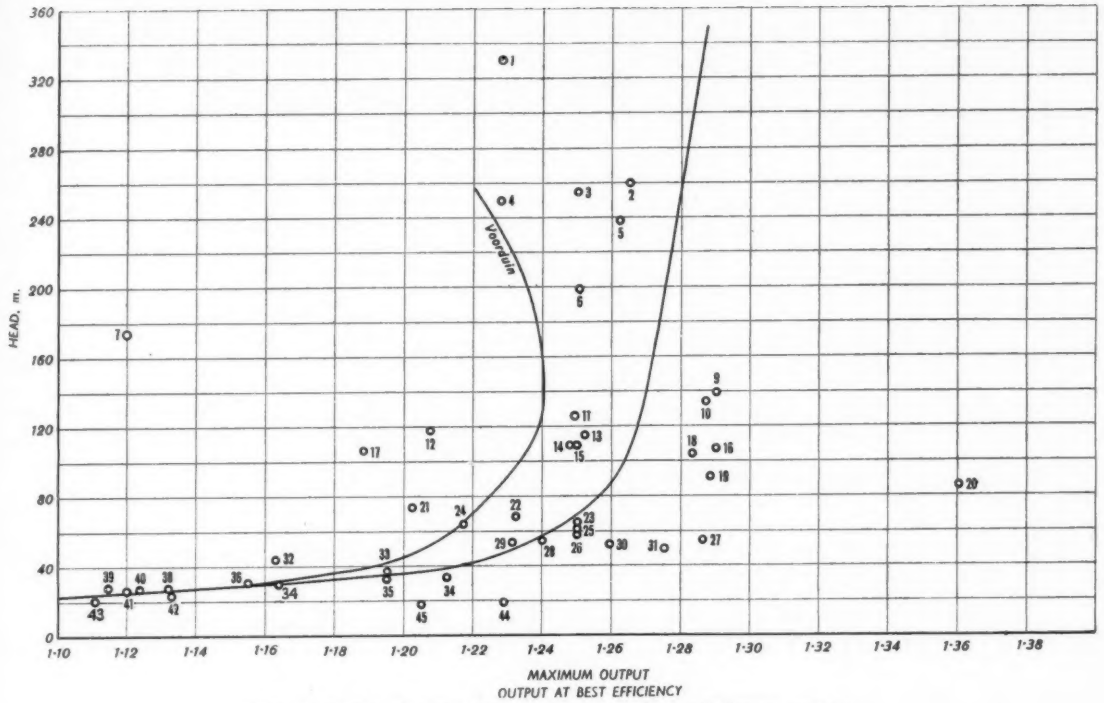

Fig. 3. Ratio of maximum output to best-efficiency output

present manufacturing technology. Besides giving the lines which enclose the surface of the plane which characterises the use of Francis turbines, the diagram also shows the limits for Pelton and for Kaplan turbines. From these it is clear that the trend is for Francis types to be used at progressively higher heads, thus invading the field up to now reserved for Pelton wheels and abandoning that of the lower heads to the Kaplan types.

Pelton wheels of a considerable output have been used down to 340–350 m. (1,030–1,070 ft.) at Lotsch, Austria (54·5 MW at 1,150 ft.) and at Bridge River, British Columbia (46·5 MW at 1,120 ft.) (both vertical six-jet type), probably because of the high solid content to be found in the water at these places; but with these heads Francis turbines are nowadays exclusively employed.

Pelton wheels have also been used at lower heads for smaller output in many well-known though somewhat older plants: at Maipo, Peru (13·7 MW at 675 ft.) built in 1934 and at Cardano, Italy (10·8 MW at 500 ft.) built in 1926.

At the other end of the range we find the first Kaplan turbine, that of Bort-Rhue, France, of 1953 at 230 ft. (23·6 MW), and they become frequent before we reach 50 m. We may also mention that of Lasele, Sweden, 65 MW at 172 ft. of 1954 and that of Pollaphuca, Ireland, 18·7 MW at 166 ft. of 1952, and even some of smaller output, such as the Italian plants of Barcis, 8·5 MW, 191 ft., and of Rocchetta of only 5·6 MW at 188 ft.

The biggest Francis turbine appears to be that at Stornorrfors built by Nydqvist & Holm A.B. in Sweden in 1955, 147 MW at 240 ft. Also of considerable output is that at Boulder (now Hoover) Dam, 116 MW at 590 ft., built by Allis-Chalmers Mfg. Co. the same year. Since 1941 the highest output had been that of the Grand Coulée: 110 MW at 330 ft.; also worthy of mention amongst the lower-head plants is that of Dnieprostroy, of 73·6 MW, 124 ft., built in 1935. Both the latter machines were made by Newport

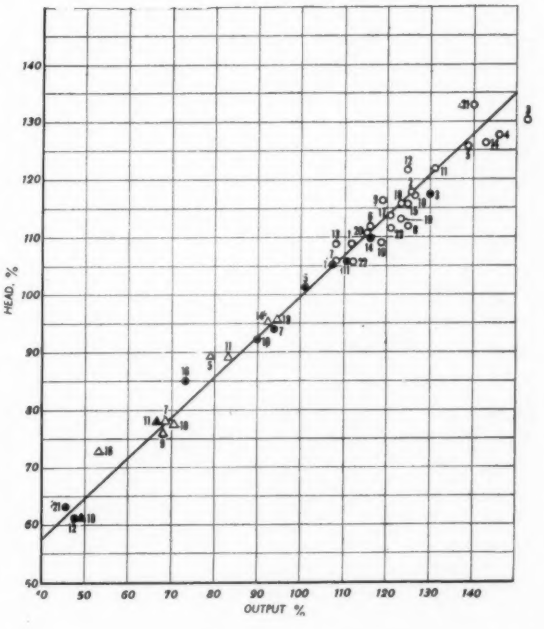

Fig. 4. Percentage of rated output with varying head

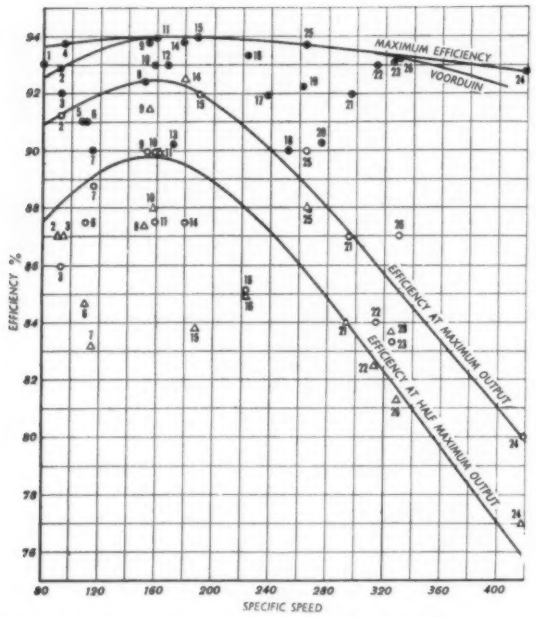

Fig. 5. Efficiencies of turbines of different specific speed

News Shipbuilding and Dry Dock Co. Notable machines will also be those at Tokke, Norway, made by Kvaerner Brug, 110 MW at 1,240 ft.

For some years the higher heads have exceeded 1,300 ft., the highest of all being that at Fionnay, Switzerland, 46 MW at 1,553 ft. A still higher head will be that of the turbines at Hemsil I, Norway, to be built by Kvaerner Brug, 35·5 MW at 1,680 ft.

Specific Speed

The relation between rated head and the most convenient specific speed is a fundamental one since it is the starting point for the actual design.

Fig. 2 which synthesises computations made for over 350 turbines is a confirmation of the proportionality between specific speed and the square root of rated head as given in all textbooks, though with various coefficients of proportionality. Curves given by many authors agree but the relation given by the "Hydroelectric Handbook" as $n_s = [6690/(H+10)]+10$ is not in agreement with actual designs. To allow comparisons between the different curves we have drawn a curve which appears to be the average. In Table 2 (published in the original article) the year of manufacture has been shown and the trend towards higher specific speeds appears clearly.

Maximum Output and Best-Efficiency Output

No general agreement exists on the rating of hydraulic turbines, i.e., as to whether it should be based on full-gate output or more nearly the best-efficiency output. In Europe manufacturers customarily give as "rated output" the maximum output, in America manufacturers quote an intermediate value; no standard specification exists on this subject. The matter is of a certain importance because—though all the design characteristics depend on the output at best efficiency—service characteristics are usually based on "rated output." The difference between these two outputs is not constant but increases with decrease in the specific speed, i.e., with increase in the head.

For this reason in Fig. 3 (derived from Table 3) we have given the ratio of maximum output to best-efficiency output for a number of turbines.

It should be noted that isolated points in the diagram are the result of special requirements; thus the Tallulah Falls turbines were designed to operate under a fairly constant head (164 to 176 m., 540 to 580 ft.) whereas with the turbines of Ocoee a good efficiency was required down to very low heads.

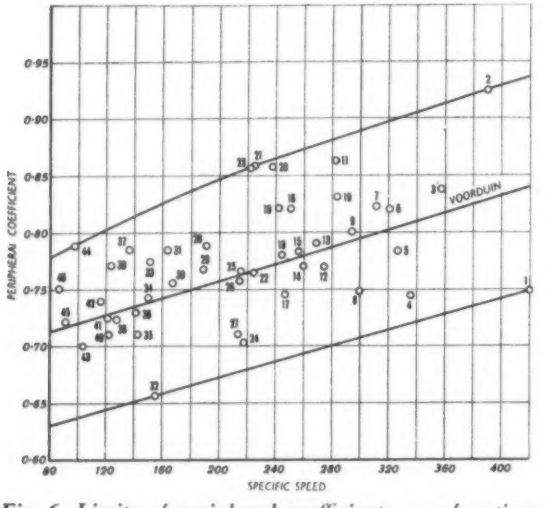

Fig. 6. Limits of peripheral coefficients as a function of specific speed

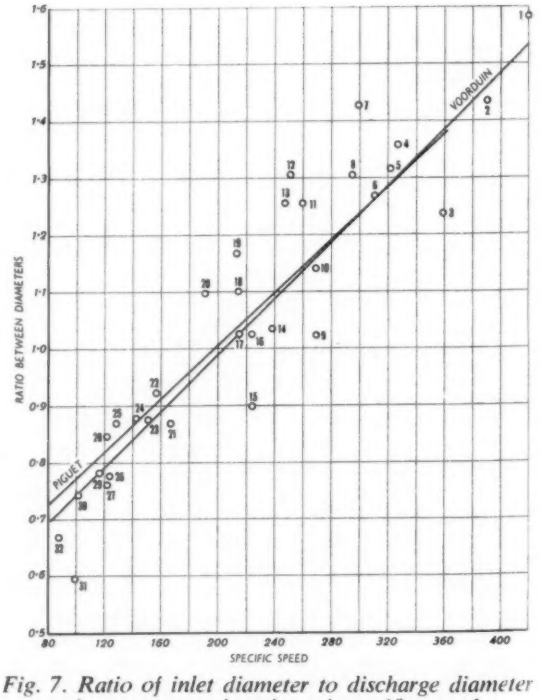

Fig. 7. Ratio of inlet diameter to discharge diameter of runner as a function of specific speed

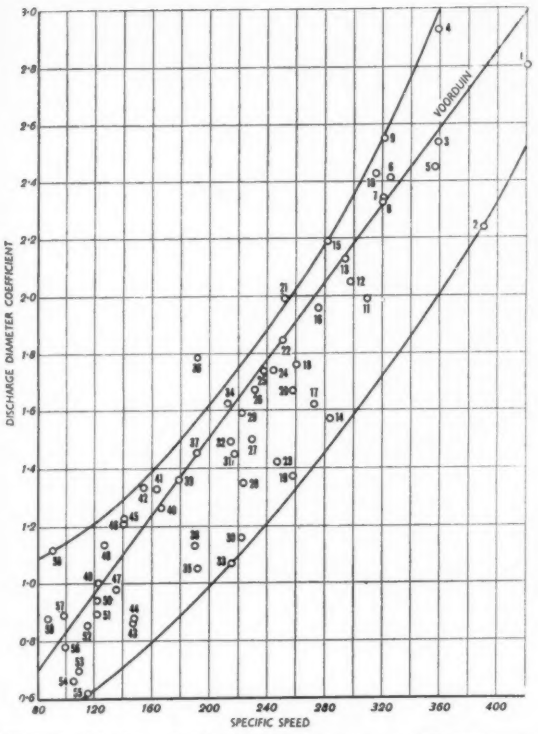

Fig. 8. Limits of discharge-diameter coefficients as a function of specific speed

Output with Varying Head

The variation of the output with head is known for 23 turbines (Fig. 4 derived from Table 4), and it shows that in the law $P = H^{\alpha}$ the influence of the variation of efficiency with the head is such that α is less than 3/2, being on the average 1·35 at heads from 100 to 133% of rated; and is more than 3/2, being on the average 1·65, at heads from 100 down to 60% of rated head.

Efficiency

Fig. 5 (derived from Table 5) shows best-efficiency. efficiency at maximum output and efficiency at one half of maximum output as a function of specific speed and as attained in machines actually built. The following observations can be made: (a) The maximum best-efficiencies are attained with specific speeds in the range from 150 to 200 and the reduction outside this range is considerable; (b) Turbines of different manufacture and even of the same manufacture may exhibit appreciable differences in efficiency. As instances, between Set No. 4 and Set No. 9 of the Otto Holden station, the former made by the Canadian Allis-Chalmers and the latter by John Inglis and of same design, there is a difference of 3 in the best-efficiency percentage. Also between Sets No. 4 and No. 5 of Des Joachims plant both made by Dominion Engineering with minor design modifications there is a percentage difference of 1·5.

DIMENSIONAL CHARACTERISTICS

Peripheral Coefficient

While the selection of a value for the specific speed is dependent on the design and on the performance characteristics of the turbine, the selection of the runner inlet diameter determines all the dimensions.

It is usual to define as "peripheral coefficient" the ratio:—

$$\emptyset_1 = \frac{\text{peripheral speed of the runner}}{\text{spouting velocity of the water}} = \frac{2\pi\frac{n}{60}D_1}{\sqrt{2gH}}$$

where D_1 is the inlet diameter of the runner measured at the distributor centre line.

Fig. 6 (derived from Table 6) gives the coefficient $\emptyset_1$ computed for some 50 turbines. It would appear expedient to select a small peripheral coefficient in order to obtain smaller diameters and lower costs.

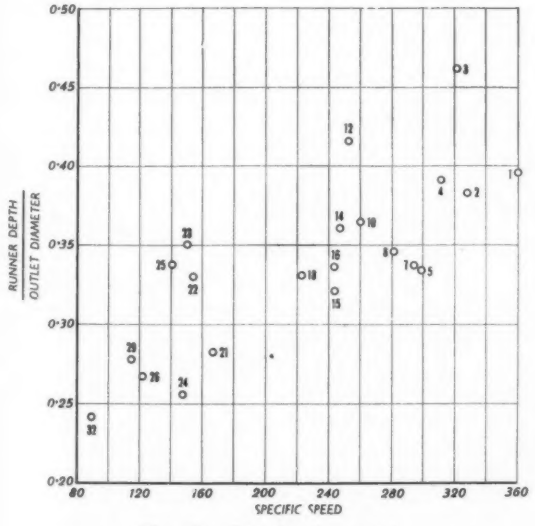

Fig. 9a. Depth of runner

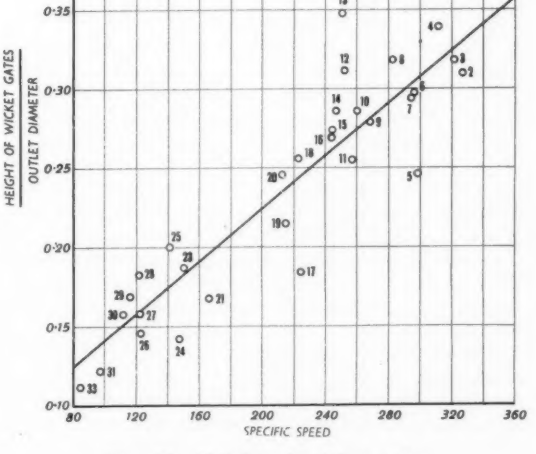

Fig. 9b. Height of wicket gates

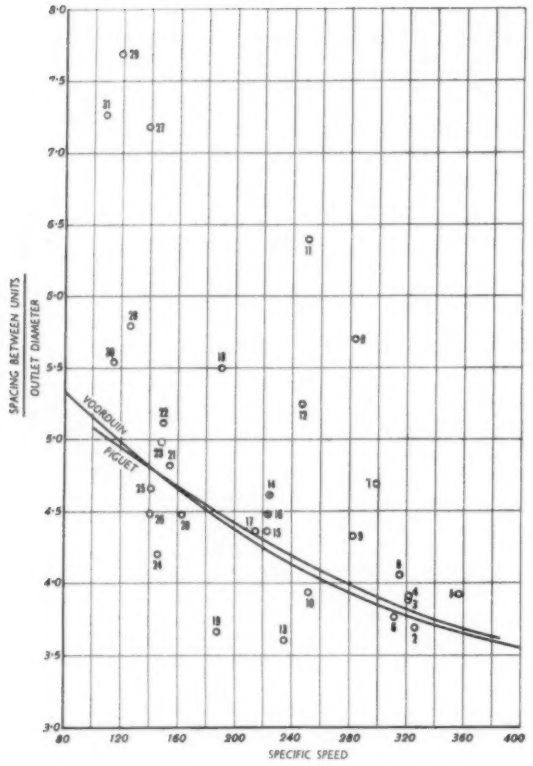

Fig. 10. Spacing of vertical-axis units

however, the higher discharge velocity which ensues produces an energy loss, i.e., a lower efficiency. It may be seen that, on the other hand, modern turbines have values of $\emptyset_1$ and hence diameters conservatively chosen and, as a consequence, reduced runner width. At lower specific speeds, i.e., for narrow runners, a high peripheral coefficient is impracticable; the desirability of high efficiency even at higher cost is offset by the danger of clogging due to insufficient width of the runner.

Runner Diameters

Once $\emptyset_1$ is selected we find the runner diameter

$$D_1 = 84{\cdot}6\ \emptyset_1 \frac{H^{\frac{1}{4}}}{n}$$

In Fig. 7 (derived from Table 7) we have also given the ratio D_2/D_1 between discharge and inlet diameters. The ratio can be larger than, equal to, or smaller than unity, depending on the value of the specific speed, the diameters being equal for speeds from 170 to 200. This can be taken as a criterion to differentiate low-speed from high-speed turbines.

An advantage resulting from selecting a ratio higher than unity is the possibility thus afforded of dismantling the runner from below without dismantling the alternator.

The discharge diameter is a more important parameter than the inlet diameter for the designer of the power house, since on the former all other powerhouse dimensions can be made dependent.

Instead of using a peripheral coefficient $\emptyset_2$ of the form already seen it is preferable to introduce the

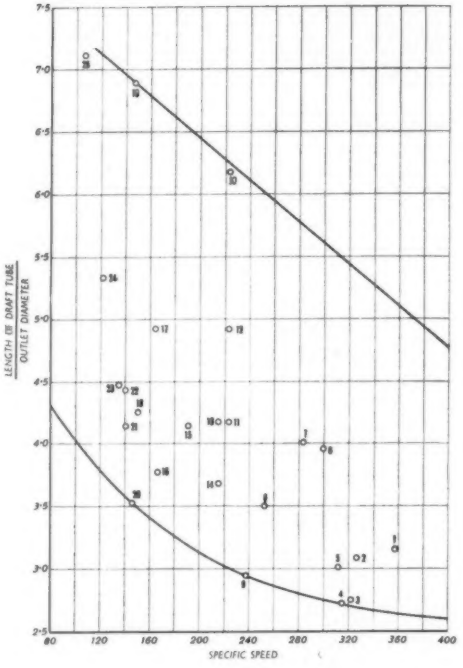

Fig. 11. Limits of length of draft tube

square root of the output $P^{\frac{1}{2}} = H^{5/4}\ n_s/n$ and to write:

$$D_2 = \frac{84{\cdot}6\ \emptyset_2}{n_s\ H^{\frac{1}{2}}\ P^{\frac{1}{2}}} = C_2\ P^{\frac{1}{2}}$$

where the new parameter C_2, defined as above, is the "discharge-diameter coefficient."

It is convenient to plot this parameter directly

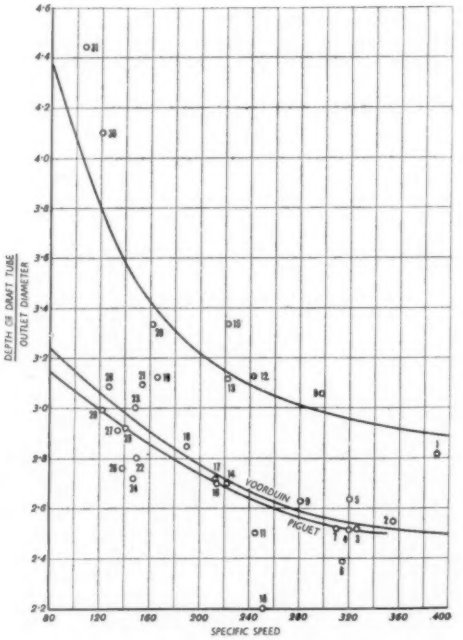

Fig. 12. Limits of depth of draft tube

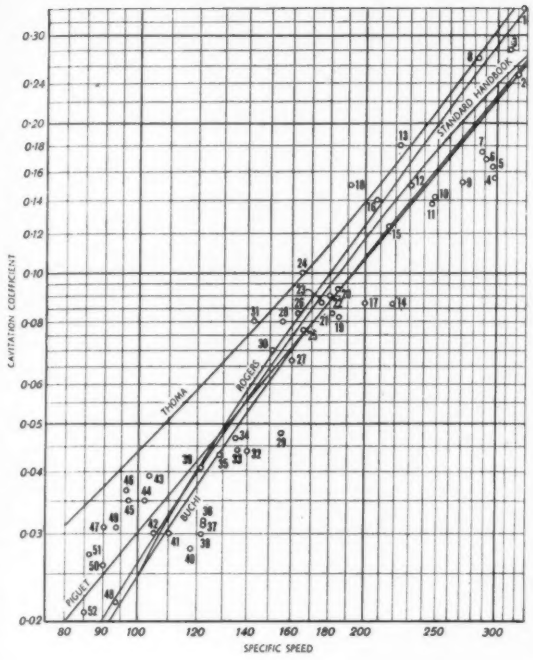

Fig. 13. Cavitation coefficients

against specific speed; the resulting curve is shown in Fig. 8 (derived from Table 8).

Height of Wicket Gates and Depth of Runner

The other parameters which determine the turbine dimensions are the height of the wicket gates and the depth of the bottom of the runner measured from the centre line of the distributor. Both dimensions are given in Figs. 9a and 9b (derived from Table 9) in terms of the discharge diameter D_2. The figures for some 30 machines of various makes are sufficiently consistent though they include some plants outside the average range.

POWER-HOUSE DIMENSIONS

Vertical-axis Unit Spacing

The figures for some 30 plants plotted in Fig. 10 (derived from Table 10) against specific speed in terms of discharge diameter show a great dispersion so that it seems difficult to draw any conclusion. The figure may be taken as an indication of some sample values only (cf. Nag, Madhavan, *loc. cit.*).

Length and Depth of Draft Tube

For these two dimensions, though the figures also show a certain dispersion, it is possible to draw limiting curves, as seen in Figs. 11 and 12 (derived from Tables 11 and 12). These allow some generalisations: Modern turbines have higher depth of draft tube. This permits a higher efficiency though at a somewhat higher cost of excavation and is the result (a) of a better knowledge of the cavitation phenomenon and hence of narrower safety margins and (b) of the use of steel and alloys of a better wear resistance.

Tailwater Level

It is well known that the tailwater level must never fall below the limit at which cavitation starts. The height of the runner with respect to minimum tailwater elevation h_s should never be higher than where:— $h_s = h_b - h_v - \sigma H$

h_b is the height of water column corresponding to barometric pressure,

h_v is the height of water column corresponding to vapour pressure,

σ is defined as the "cavitation coefficient."

It is outside the scope of this work to deal with the phenomenon of cavitation but in accordance with the main purpose we give the values computed for some 50 plants plotted against specific speed in Fig. 13 (derived from Table 13). The comparison with the best-known formulae is interesting and it confirms the validity of the formula due to Thoma.

Bibliography of Plants

The original article concludes with a bibliography covering the 350 or so plants on which the survey has been based. For each plant at least one bibliographical reference has been given, choosing wherever possible the most complete and satisfactory.

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2 *La Genie Civil*, CXXVIII (1951), p. 27.
3 *Kraft och Ljus*, 22 (1949), p. 189.
4 WATER POWER, 7 (1955), p. 210.
5 Fourth World Power Conference, 1950, mem. H/3.1.
6 *Ibid.*, mem. H/2.2.
7 *L'Energia Elettrica*, XXVII (1950), p. 510.
8 "Hydro-Electric Handbook," Wiley, New York, 1949, 2nd ed.
9 "Standard Handbook for Electrical Engineers," McGraw-Hill, New York, 1949, 8th ed.

Rosshaupten Power Station

We regret that, in part two of the article on Rosshaupten power station published on page 5 of our issue for January 1959, we omitted to state that the butterfly valves installed at Rosshaupten were manufactured by Escher Wyss, G.m.b.H., Ravensburg.

Line Conductor Order for New Zealand. Aluminium Wire & Cable Co. Ltd. have received an order, of value £253,000, from the New Zealand State Hydro-Electric Department for steel-cored aluminium conductors to be used on a 220 kV transmission line in the South Island. The conductors will have a section of 37/·146 in. and will be used to string part of the Livingston-Benmore-Islington (Christchurch) line.

Percussive Rock-Drill Bits, Rods and Accessories. A new catalogue received from Rip Bits Limited gives full details of their range of rock-drilling bits and rods. This publication is handsomely produced and is crammed with technical information.

Radiographic Equipment. A new firm, R. F. Fraser-Smith, of 69, Kings Cross Road, London, W.C.1, announce that they are able to supply radiographic equipment and accessories; in particular, they are manufacturers of Gamma ray equipment of a new design. They are prepared to deal with enquiries about non-destructive testing.

Fig. 1. General view of Salto I power house and switchyard, and intake settling basin

The Laguneta Hydro-Electric Project

By CARLOS S. OSPINA and JAIME MONTAÑA

THE Laguneta hydro-electric project, on the Río Bogotá, in Colombia, was recently completed. Three 20 MW sets are now operating and a fourth is to be placed in operation within a twelvemonth. The power will provide for the rapidly growing needs of Bogotá, capital of Colombia, and its surroundings.

This project is part of the development of the Bogotá river and has been under consideration since early in the century. Fig. 2 shows a map of the river and Table I gives relevant data. Salto II and Charquito II are scheduled for construction in the near future and will add 92 MW to the system.

Empresas Unidas de Energía Eléctrica de Bogotá control all the public power facilities in Bogotá and vicinity, and own all the power plants on the Bogotá river. This concern was founded by private enterprise in 1896, but in 1927 the city of Bogotá acquired a controlling interest and in 1951 bought up the entire stock. It is run by an autonomous Board.

Bogotá is at present served, in addition to Laguneta, by two hydro and one steam station with a total effective peaking capacity of 74 MW. The population approaches one million and the use of power has had to be restricted for several years.

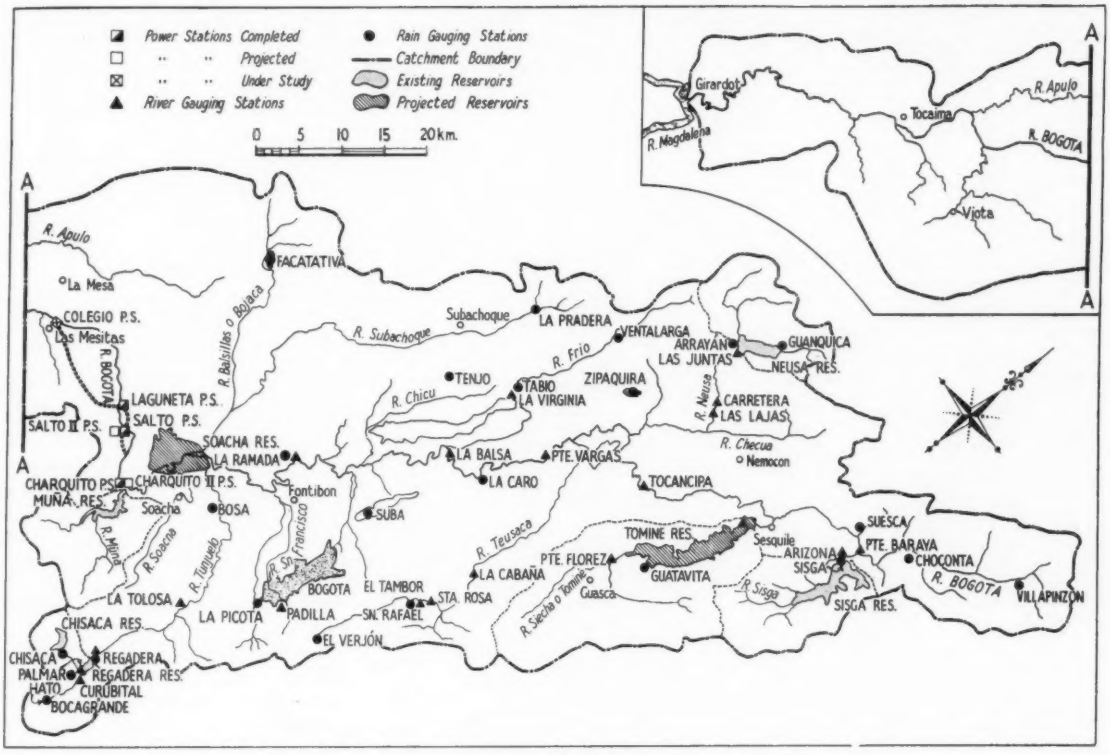


Fig. 2. Hydrographical map of River Bogotá run-off area

The Bogotá River

The utilisation of the Bogotá river has the following characteristics all of which have influenced the development of its power resources:—

1. Ample storage facilities exist in the upper basin. Three important reservoirs have been built (Sisga, Neusa and Muña) and two more are under investigation (Tominé and Soacha).

2. An ample supply of coal exists for steam-electric generation.

3. No storage is available below Alicachín and there is very limited pondage between each development.

4. There is a site for a pump-storage reservoir at Muña and a pondage reservoir at Alicachín, capable of regulating the flow for all the power plants.

5. There are very advantageous topographical conditions for power development, but very difficult geological formations, particularly below Laguneta.

Stream flow

The upper Bogotá River basin has remarkable hydrological features:

Area (above Alicachín)	4,270 sq. km.
Mean yearly rainfall	865 mm. (1934–1954)
Mean flow	25·6 cu. m. per sec. (1934–1954).
Mean run-off coefficient	0·22

The outlet of this basin is at Alicachín, but the flow has been gauged at the Salto intake. There are no appreciable tributaries between Alicachín and the Laguneta intake. Stream flow records are available from 1934 and rainfall records for the city of Bogotá have been taken since 1866.

In the dry season, due to the use of water for irrigation, the minimum recorded flow of the Río Bogotá has been as low as one cu. m. per sec. With the present storage facilities the flow should not fall below 14 cu. m. per sec. and with the proposed storage facilities available the flow could be regulated to 21 cu. m. per sec. Flood peaks are low, the maximum recorded flood at the Salto intake did not exceed 120 cu. m. per sec. The Laguneta project has been designed for a normal peak flow of 34 cu. m. per sec. and for a maximum flow of 36 cu. m. per sec.

Engineering Studies

Planning of the overall development of the Río Bogotá was originally carried for the Empresas by Motor Columbus of Baden, Switzerland. Dr. Edgar Jeuch was in charge of this work from 1945 to 1956 and it was he who developed the basic scheme for Laguneta.

The preparation of the constructional drawings for the Laguneta scheme, the purchase specifications, the design of the main transmission line to Bogotá and the supervision of erection work were effected by GAI Pan American Corporation, a subsidiary of Gilbert Associates of Reading, Pennsylvania, U.S.A.

The latest investigations on stream flow, regulation, storage and general planning and the design of the new Alicachín pondage structure were accomplished by Olarte, Ospina, Arias & Payán, Ltd. (OLAP) and Tippetts-Abbett-McCarthy-Stratton (TAMS). These firms are also doing the engineering work required for the next two power plants on a Bogotá river, the Salto II and Charquito II.

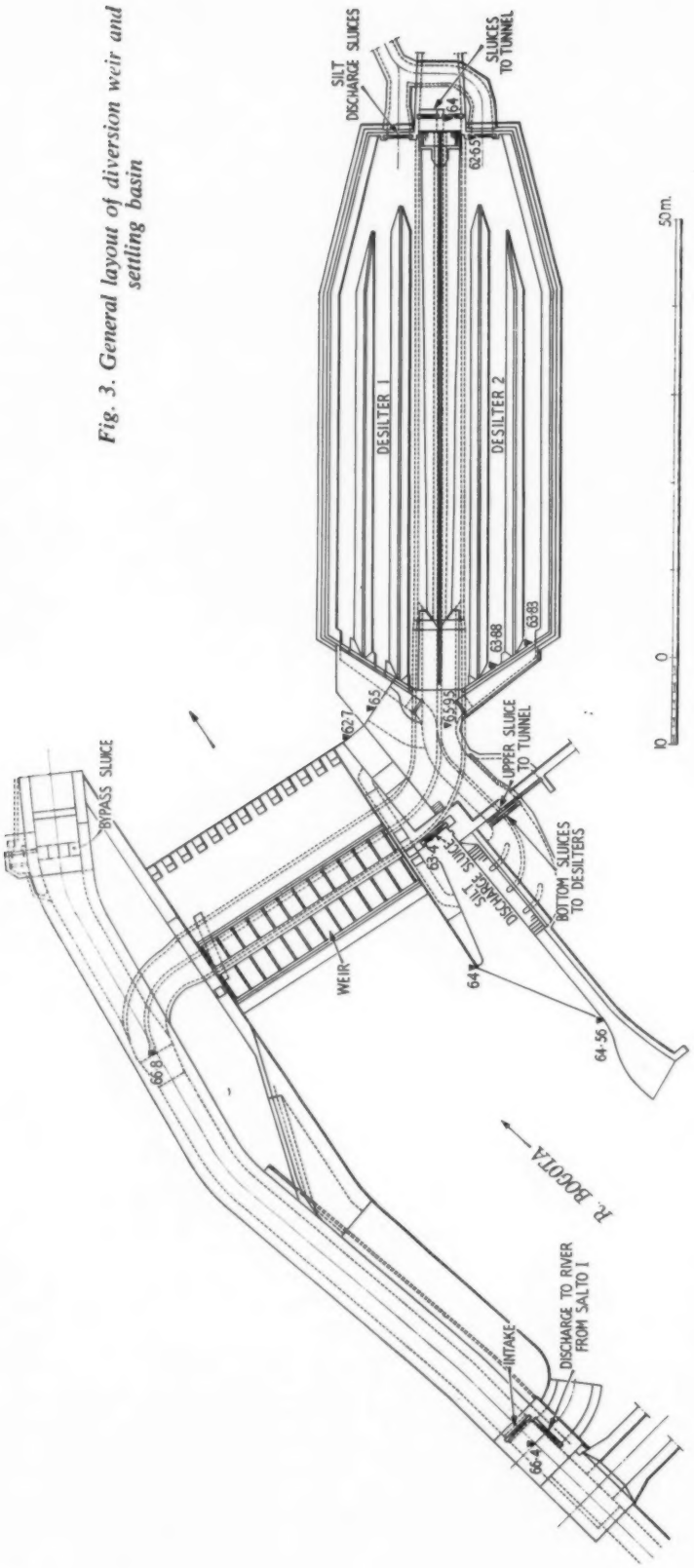


Fig. 3. General layout of diversion weir and settling basin

Geological investigations for the Laguneta project were directed by Dr. Enrique Hubach and by Mr. Cecil B. McGavock, Jr.

Laguneta First Stage

The initial installation at the Laguneta plant comprises three sets of vertical Francis turbines with direct coupled generators. This installation has a total capacity of 60 MW with a water consumption of 25·5 cu. m. per sec. It was initially planned that at least one unit of the plant would be in service by the end of 1956, but owing to delays experienced in negotiating contracts and in the delivery of equipment, completion of the first stage was not accomplished until early in 1957. Fig. 6 shows a general plan and profile of the Laguneta project. Its main features are as follows:—

Intake and Pondage Capacity. The upper components of the project include a diversion dam and settling basin located immediately below the existing Salto I plant. These upper works, with a combined storage capacity of 16,000 cu. m., are so designed that, under normal operation, the de-silted water from Salto I tail-race will by-pass the Laguneta settling tank and be carried directly into the Laguneta conduit system. Water can also be taken from the river at the intake and passed through the settling basin to provide for the water requirements of Laguneta when in excess of the existing flow through Salto I. In the event of a complete shut-down of the Salto plants, the Laguneta plant can continue in operation by utilising the intake and settling basin to full capacity.

Fig. 3 is a general layout plan of the diversion dam and settling basin. The diversion dam consists of a bear-trap gate with a crest length of 25 m., which automatically controls the water surface of the river at a predetermined level. The sluice gate consists of a vertical-lift gate 3 m. high and 4·70 m. wide with an overflow protective plate for a top discharge.

Pressure tunnel. Extending from the settling basin and intake is a concrete-lined tunnel, 3·30 m. in diameter and approximately 2,200 m. in length, with a connecting branch tunnel to a regulating basin of 43,000 cu. m. of storage capacity. This basin will be completed in the second stage of the project.

Fig. 4. View of intake; main gate in lowered position

A typical section giving details of the supports, shuttering, lining and reinforcement used in the pressure tunnel is given in Fig. 5. Only a small proportion, about 8%, of the total excavated length was not supported. From the surge tank downstream to the valve house, a distance of 184 m., the reinforced concrete lining was supplemented by a steel liner 3·15 m. in diameter and 10 mm. in thickness. The grouting of the tunnel was carried out in two stages. During the first stage low pressures of between 50 and 100 lb. per sq. in. were applied but in the second stage the grout was injected at pressures of between 200 and 300 lb. per sq. in. The quantity absorbed per linear metre of tunnel normally ranged between two and three cu. m. with a minimum of 0·7 cu. m. in good terrain and a maximum of 6 cu. m. in bad terrain. This grouting treatment was effective in preventing any appreciable leakage from the tunnel. Construction of the tunnel started in September 1954 and it was completed in May 1957.

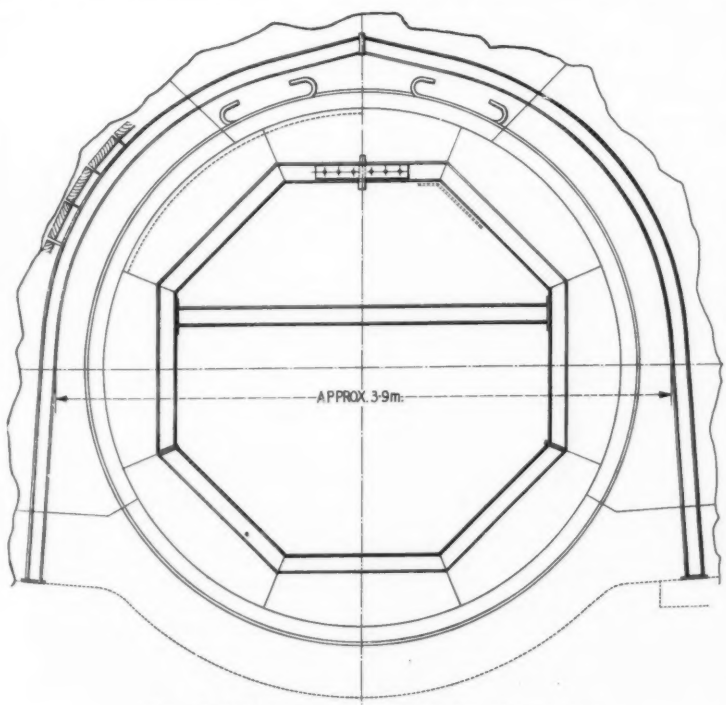


Fig. 5. Main tunnel with details of supports

The surge chamber is of the restricted orifice type, 30 m. high and 7 m. in diameter. It was excavated from the surface and the concrete lining was inserted as the sinking proceeded. The tunnel connects, through a steel Y-piece with two steel penstocks approximately 912 m. in length and having an average diameter of 2 m.

The valve house occupies an area of 15 m. by 9 m. Each penstock is served by two butterfly valves, one of which is hand operated and the other weight operated. Each penstock is also equipped with an air valve located immediately downstream of the butterfly valves.

The penstocks are located above ground and convey the water to the power house through a drop of approximately 290 m. On the upper part, for a length of 336 m.

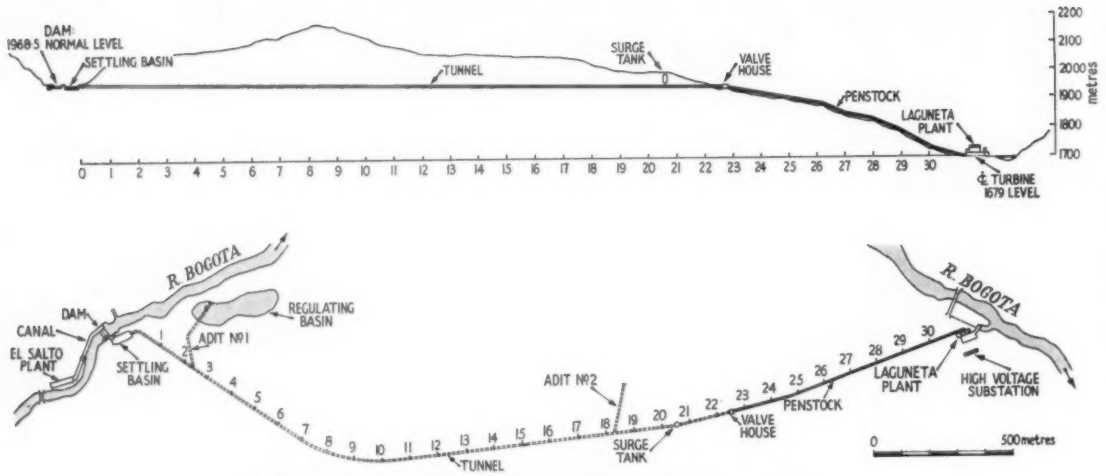


Fig. 6. General plan and profile of Laguneta project

they have a diameter of 2·10 m. with steel plate thickness varying from 10 mm. to 13 mm.

Along the lower 543 m. the internal diameters are 2·00 m. and 1·90 m., and the steel plate thickness ranges from 12 mm. to 24 mm. The penstocks are installed on concrete saddles.

At the lower end each penstock bifurcates, each branch being 1·50 m. in diameter and equipped with Smith ball valves 1·02 m. (40 in.) in diameter. The middle generating unit is served by two of these branches, as shown in Fig. 8.

Power House and Generating Equipment

The power house is a conventional fully enclosed structure with separate floors providing for: (a) turbines, valves, shop area and assembly; (b) main generators, regulators, auxiliaries and service equipment, and (c) electrical control room, offices and annexes. The generator hall is equipped with a 50 ton travelling crane including an auxiliary hook for 7·5 ton service. On one side of this hall there is an open pit which allows the crane to reach the turbine valves.

The three vertical turbines were manufactured by S. Morgan Smith, and each develops 30,100 h.p. at full gate at a speed of 720 revs. per min. under a net head of 281 m. The maximum runaway speed guaranteed not to exceed 1,140 revs. per min. under a net net head. The runner is made of cast stainless steel and the $13\frac{1}{2}$ in. diameter shaft of forged open-hearth steel. Provision has been made to remove the runners from beneath and to take them, as well as any other removable part of the turbine, out through the turbine pits without disturbing the generators. Each turbine is equipped with a Woodward governor of the HR gate-shaft type. The governor operates by oil pressure supplied by a motor-driven pump and there are means for adjusting the speed directly, as well as by the speed-control motor.

The three generators were supplied by Canadian General Electric, vertically mounted, Y connected and are rated at 22,500 kVA, 720 revs. per min. three phase, 60 cycle, 6·900 V at 0·8 power factor. These generators are designed for a maximum temperature rise of 60°C. based on an ambient temperature not exceeding 40°C. and for an altitude of 1,700 metres (5,600 ft.) above sea level. The rotating parts of the generators are designed for a moment of inertia WR^2 of 470,000 lb. ft.^{2}

Each generator is provided with an enclosed cooling system complete with air ducts, surface coolers and metal housing. Circulation of cooling air through the ventilating system is accomplished by means of fans on the generator rotor. The water entering the cooler has a temperature below 20°C. and an exit temperature not exceeding 25°C. The cooling water is obtained from a water treatment plant.

Fig. 7. Further view of weir and intake

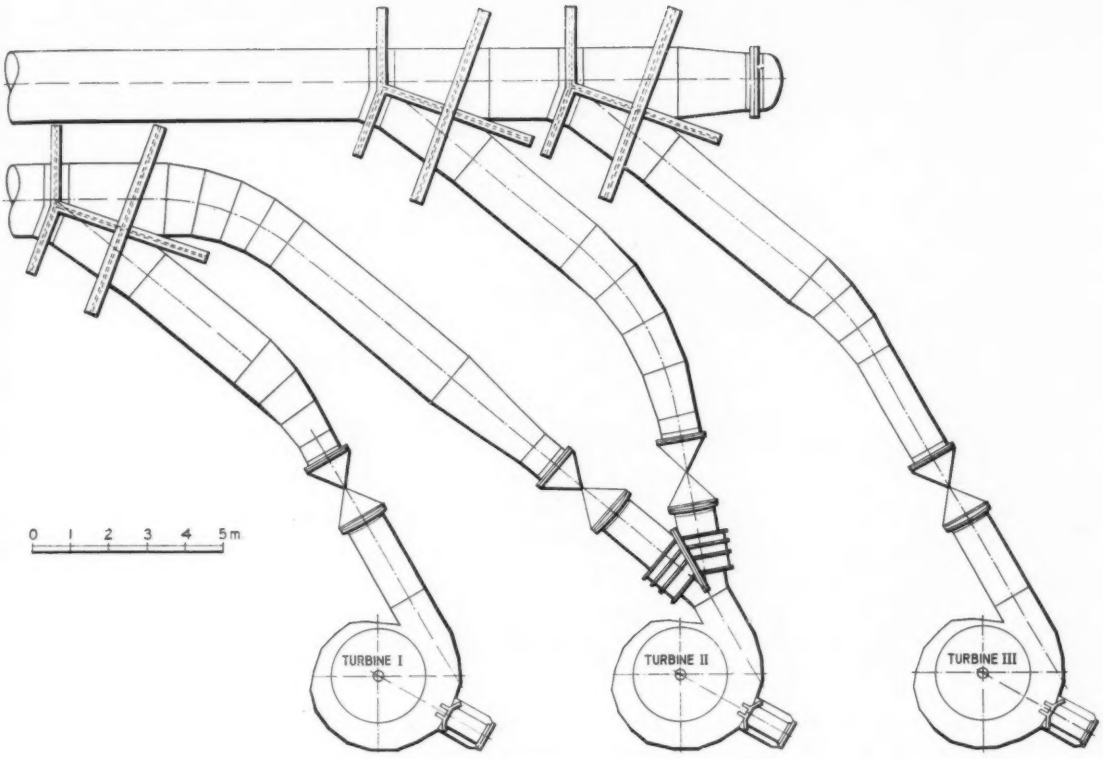


Fig. 8. Details of penstocks at power house

A total of ten single-phase AEG transformers are in service and are distributed in three banks of three transformers each, plus one spare. Each is rated at 9,375 kVA, 60 cycles, 6·9/57·5–115 kV, delta-star connected and earthed through an earth-fault neutraliser at 57·5 kV (115 kV future).

Fig. 9. Interior of pressure tunnel

Switchyard

The outdoor switchyard consists of a double 115 kV bus arrangement on a five-bay steel structure. The buses are laid vertically, the transfer bus being located in the upper part of the structure. At the present time it includes three oil-circuit breakers, for the three transformer banks, operating at 57·5 kV with provisions for future operation at 115 kV, and one bus-tie circuit breaker and three transmission line circuit breakers.

With the exception of the bay containing the bus-tie circuit breaker, equipped with only two disconnecting switches, all the bays house two oil breakers, each equipped with two OCB disconnecting switches and one air-break disconnecting switch.

The seven circuit breakers are of the triple-pole, single-throw oil type suitable for operation on a 57 kV, 60 cycle, 3 phase, neutraliser-earthed system with a 3-phase short-circuit stress of less than 1,750 MVA. The breakers will also be capable of future operation on a 115 kV system with a short circuit stress of less than 3,500 MVA. The breakers are rated for continuous operation at 115 kV and 1,150 A, having an interrupting rating of 3,500 MVA (three phase) and 17,500 A (rms) at rated voltage, for operation at 9,000 ft. The transmission lines are provided with carrier-current devices rated 69,000–115 V for telephone communication. The lines are also provided with wave traps, and lightning arresters.

Coupling capacitor potential devices without carrier are also installed on the switchyard 57·5 kV main and transfer buses, for synchronising. Instrument

TABLE I.—SUMMARY OF EXISTING AND IMMEDIATE PROJECTS ON THE BOGOTA RIVER

ITEMS	EXISTING PROJECTS						IMMEDIATE PROJECTS	
Plant	Salto I		Charquito I		Laguneta		Salto II	Charquito II
	Units 1 to 5	Units A & B	Units 1 & 2	Units 3 to 6	Units 1 to 3	Unit 4		
Description								
Type of plant	Hydro	Hydro	Thermal	Hydro	Hydro	Hydro	Hydro	Hydro
Number of units	5	2	2	4	3	1	2	2
Capacity (MW)	50	4·5	14·5	5·7	54	18	66	26
Head (m.)	419	400	—	48	287	287	417	62
Max. flow for which designed (cu. m. per sec.)	15·4	1·8	—	18	36	8·5	21	53
Waterways								
Tunnel: length (km.)	1·6	2	—	—	2·1	—	1·7	2·0
diameter (m.)	2·4–2·3	—	—	—	3·3	—	2·65–2·40	4·5–4·3
Penstock: number	2	1	—	2	2	—	1	2
length (m.)	727/715	700	—	364	837	—	700	110
diameter (m.)	1·58–1·20	0·9–0·8	—	1·72/1·80	2·10–1·9	—	2·3–2·05	3·04–2·84
Flow for each penstock (cu. m. per sec.)	7·7	3·0	—	9	18	—	21	26·5
Turbines								
Type of units	Pelton	Pelton	Steam	Francis	Francis	Francis	Pelton	Francis
Total capacity (1,000 h.p.)	66·5	6	19·4	7·6	83·1	27·7	90	33
Unit capacity (1,000 h.p.)	13·3	3	—	—	27·7	27·7	45	16·5
Speed (r.p.m.)	600	900	3600	600	720	720	400	400
Generators								
Total capacity (MVA)	62·5	56	18·2	7·4	67·5	22·5	81	32
Unit capacity (MVA)	12·5	28	10/8·2	—	22·5	22·5	40·5	16
Generator voltage (kV)	6·6	0·4	6·6	6·6	6·9	6·9	13·8	6·6
Power factor	0·8	0·8	0·8	0·8	0·8	0·8	0·8	0·7
Substation installation								
Capacity (MVA)	65	7	14	7	84·3	28·1	84·0	34·0
Voltage range (kV)	6·6–57	—	6·6–57	6·6–57	6·9–57·5/115	6·9–57·5/115	13·2–57·5/115	6·6–57·5/115

Fig. 10. Penstock, power house and switchyard

transformers on main bus and incoming and outgoing lines are used for instrumentation, metering and relaying.

The main transmission to Laguneta is by means of a double circuit 115 kV line, 36 km. long, which feeds into the Salitre substation near Bogotá. The line is supported by 130 galvanised latticed-steel towers. The six conductors are 605,000 circular-mill ACSR.

The distance between towers varies, but in general they are about 250 m. apart. There is a span of 1,248 m. near the Salto de Tequendama where the rough terrain makes this possible.

There is also a single circuit 115 kV line interconnecting the Laguneta and Salto power plants. It has

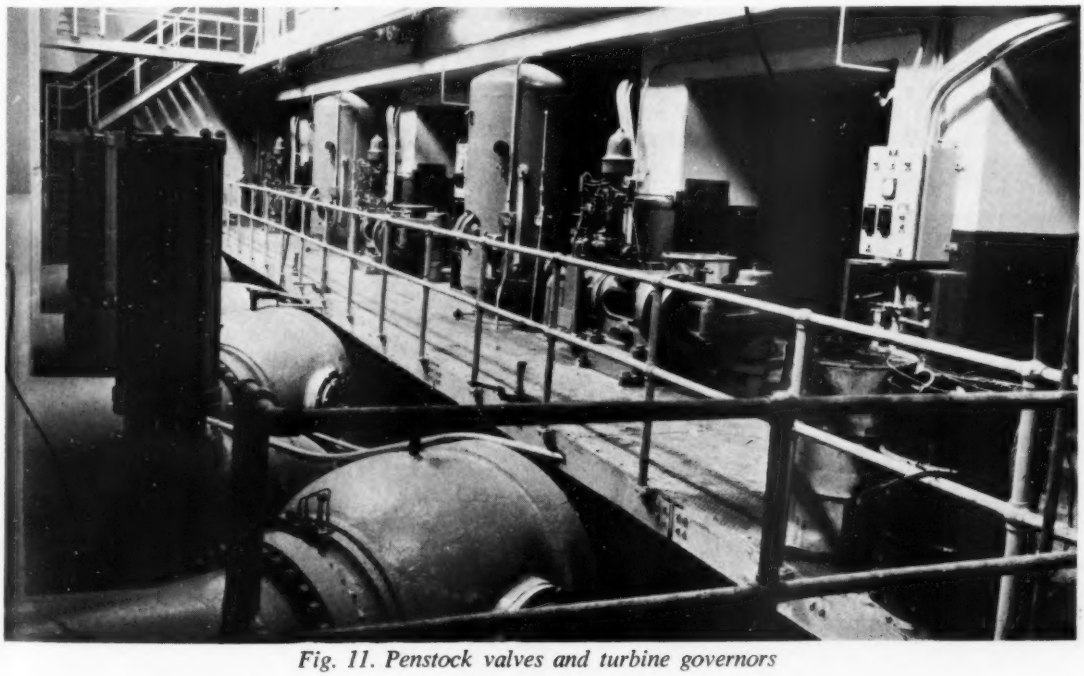


Fig. 11. Penstock valves and turbine governors

a length of about 3½ km. and is supported on 13 towers.

Water for the service of the Laguneta power plant is pumped from the Bogotá river and then clarified and filtered in a treatment plant. This water is suitable for domestic use and also for cooling purposes. The cooling-water circulates on a closed system with make-up added from the treatment plant. A cooling tower is located near the end of the tail-race channel and was supplied by the Phillips Cooling Tower Company.

Service power is available from an 11·4 kV transmission line connecting with the Salto stations. For emergencies a diesel plant is available.

Major Equipment Suppliers. The major equipment components for the project were supplied by the following manufacturers:

Item	Supplier	Nationality
Intake gates—misc.	Voith	Germany
Intake automatic control gate	Buss	Switzerland
Steel lining for tunnel and penstocks	M.A.N.	Germany
Butterfly valves	Voith	Germany
Turbine valves	S. Morgan Smith	U.S.A.
Turbines	S. Morgan Smith	U.S.A.
Electric generators	General Electric	Canada
Switchboards and station controls	Westinghouse	U.S.A.
Switchboards and station controls	General Electric	U.S.A.
Step-up transformers	A.E.G.	Germany
High-voltage breakers	Westinghouse	U.S.A.
Power-house crane	Manning, Maxwell & Moore Inc.	U.S.A.
Switchyard structure	Westinghouse	U.S.A.
Transmission towers	Lehigh Steel Co.	U.S.A.
Transmission line conductors	Alcoa	U.S.A.
Transmission line insulators	Ohio Brass Co.	U.S.A.

Contractors. Constructional work on the first stage of the Laguneta plant and related installations was carried out by the Frederick Snare Corporation of New York, who acted as general contractors. The contractors for the erection of transmission towers and the installation of circuits from Laguneta to Salto and from Laguneta to Bogotá were Cuellar Serrano Gómez y Cía. Ltda., & Ingeniería y Construcciones Ltda. The installation of the power house electrical equipment was carried out directly by the engineers of the Empresas.

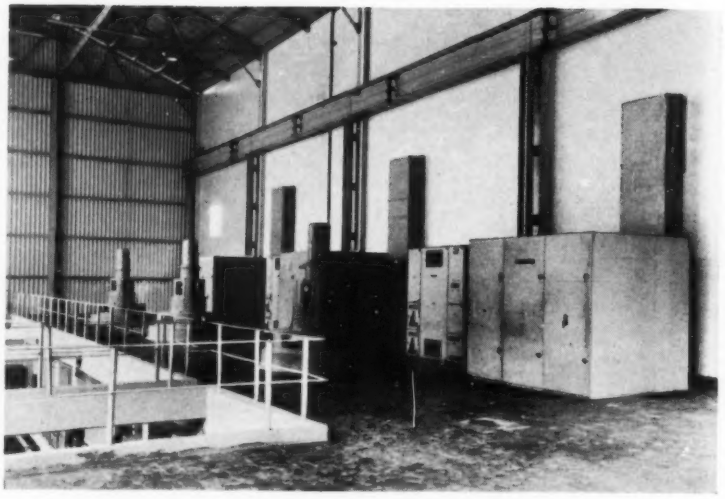


Fig. 12. Main floor of power house

Management. Management of the project was under the direction of Dr. Manuel Madero París, General Manager and of Dr. Darío Valencia, technical director of the Empresas Unidas de Energía Eléctrica de Bogotá.

Cost and Financing. The total cost of the first stage including the double-circuit transmission line of 115 kV. from Laguneta to Bogotá was 53 million pesos. The construction of the project was financed directly by the Empresas, no long-term loans being required. Short and medium-term loans were arranged with local banks.

Originally the investment for the construction of Laguneta was established on the basis of a 60% with the Empresas resources and a 40% with an external loan operation with the International Bank for Reconstruction and Development. This loan was studied and approved in principle by the Bank, but its application was suspended in the last months of 1956, when the construction was very much advanced. Since the suspension of the project would have affected seriously the development of the city of Bogotá, the Empresas decided to continue the work, recurring to internal loans with local banks.

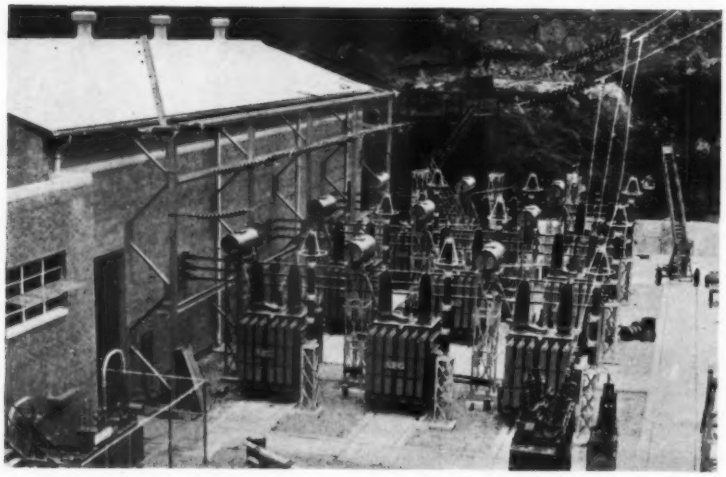


Fig. 13. Switchyard

Turbine Efficiency Measurement

A summary is given of the better-known methods of determining turbine efficiency for the benefit of the non-specialist engineer

PART THREE

Pitot Tube

The Pitot tube will need little introduction to the average engineer because this instrument has been associated with the measurement of both gas and liquid flows for very many years. Pitot himself was the first to make a total, as well as a combined, total-static pressure probe for velocity measurements in river flow and to consider in great detail possible applications for his invention.

This instrument is not normally used for measuring low-velocity flows because it then tends to be inaccurate, but it is particularly suitable for measurements in closed conduits, provided the velocities are not excessive and that the instrument is rigid enough to avoid vibration. For turbine efficiency tests, the measuring circuit consists of an impact tube linked over a suitable differential manometer to a static pressure indicator. In the case of a combined instrument, the latter consists of a number of static pressure orifices arranged in ring form on the instrument head. If an impact tube only is used, it is common practice to deduce the static pressure in the test section from suitably positioned wall static tappings. The traverse arrangement by which an impact tube may be moved to different positions on a diameter, a minimum of two right-angle traverses being specified by the power test codes, is sometimes replaced by a rake of permanently mounted impact heads. The datum static pressure is again obtained from wall static tappings arranged in interference-free position in the vicinity of the rake, which in general takes cross form. One of the advantages of such an arrangement is that load fluctuations affect all readings simultaneously, in contrast to traverse readings where load changes may occur during the time taken for a complete traverse.

Combined instruments, probes which read both impact and static pressure, are often designed for direct indication of velocity pressure, their coefficient being unity at adequately high velocities of flow. In the case of single impact tubes, the coefficient is taken according to power test codes to vary as a function of the pipe factor, the ratio of mean to centre line velocity, that is to say, of Reynolds number.

In the modified version of the Pitot tube known as the pitometer, the tubes are mounted in a rod which is inserted into the pipe through a coupling gland and valve. There is also a reversible type in which a light flat rod is used, which at its end features two bend tubes that can be turned by external control to lie in one plane; their open ends, the pressure orifices, then face into and away from the flow and the rod itself can be turned if a check on accuracy or direction of flow is required.

The tubes are connected to differential manometers. Specific gravity is usually measured *in situ* by balancing the liquid used in the manometer against water in the disconnected U-tube. For turbine-efficiency tests, the rod is kept with the orifice at the centre of the pipe to establish a point of reference, and it is then moved from point to point across the pipe, readings being taken as the means of possible manometer fluctuations. The purpose of this operation is to determine the distribution of velocity in the pipe and, consequently, the pipe factor. The differential head is the difference of level between the water columns supported by the pressures inside the orifices. The deflections of the differential manometer are multiplied by the specific gravity of the liquid used, reduced by one.

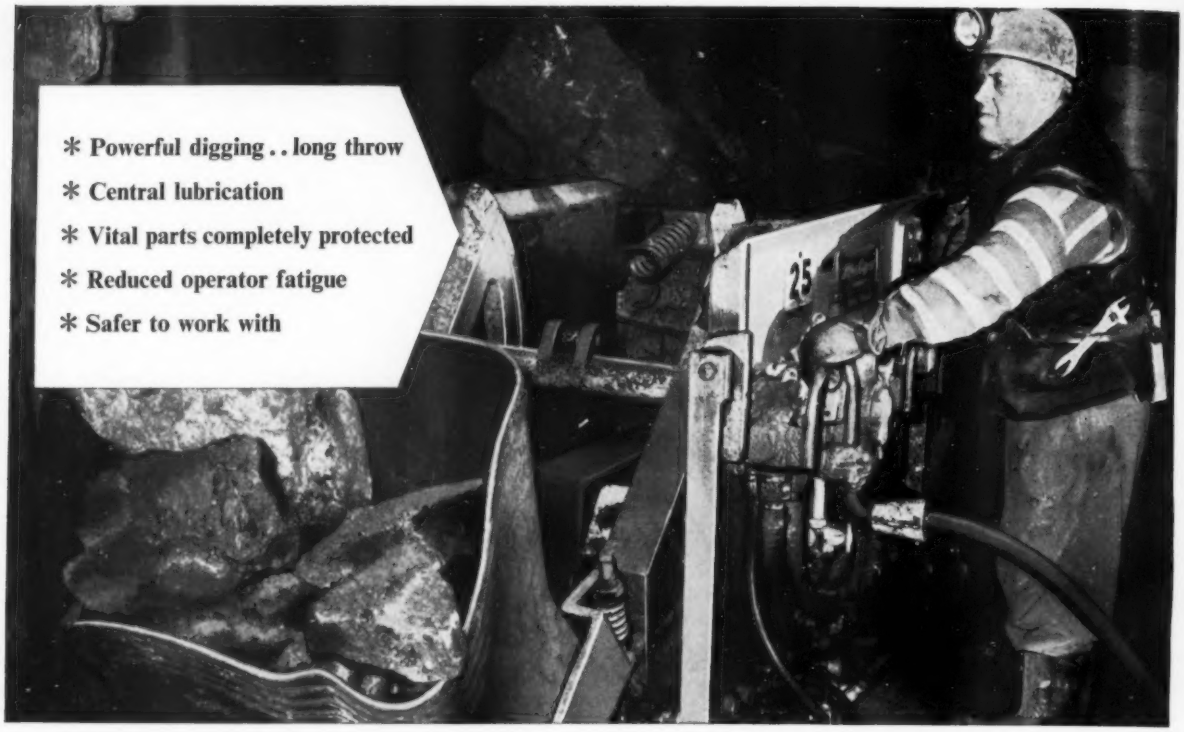
The basic expression for velocity of flow with the Pitot tube is:

$$V = K\sqrt{2gH}$$

Where V is the velocity of flow in ft. per sec., K is the instrument coefficient, g is the acceleration due to gravity in ft. per sec. and H is the differential head between static and impact pressures in feet of water. Before the tube can be used, the value of the coefficient K must be determined experimentally, and this is accomplished by rating or calibrating the instrument by towing it through still water in a rating channel, or by placing it in a uniform current; in both cases, the procedure is the same as for current meters. Under suitable conditions, the value of the coefficient is nearly always 1·00, irrespective of the velocity and of the shape of the impact orifice, so long as the Reynolds number based on the outer diameter does not fall below 2,000 for a combined Pitot-static tube.

A Pitot tube only gives a measure of the velocity of flow at the point where its orifice is situated and it is therefore necessary to relate the mean velocity to the velocity at the Pitot-tube orifice if the volume of flow is required. For the application of turbine-efficiency measurement, where only circular pipes are encountered, the velocity distribution is a function of Reynolds number. As already stated, it is usual for the Pitot-tube orifice to be located in the centre of the pipe because the velocity distribution curve is flattest at this point, and thus if the tube is inaccurately placed, smaller errors in registration are likely to arise. In addition, under steady flow conditions, the maximum differential head is generated at the centre. This procedure is based on the assumption that the single reading on the pipe centre line represents the total flow through the section.

Denoting the value of the centre velocity by V_c, and the mean velocity by V_m, $\frac{V_m}{V_c} = 0{\cdot}5$ if the Reynolds number is lower than 2,000, in which case the flow


* Powerful digging . . long throw
* Central lubrication
* Vital parts completely protected
* Reduced operator fatigue
* Safer to work with

LM56 SHOVEL LOADER *loads 65 cu. yds. an hour*

Digs Deep . . . fills long cars to capacity

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Central lubrication simplifies maintenance

Synchronized to the working tempo of the loader, the central lubricator supplies the right quantity of oil to the right part of the machine at the right moment. The daily control can thus be confined to one lubrication point only. Oil delivery is easily regulated by a screw on top of the lubricator and checked through a glass tube.

Vital parts completely protected

LM56 is not affected by the water and dirt that surround it underground. The vital parts are completely protected by special sealings, and the amply proportioned gears are made to withstand the stresses of rough work. That is why it gives long and reliable service under the toughest conditions.

Special silencers and new controls reduce operator fatigue

LM56 is remarkably silent running and easily operated. Each motor is fitted with a separate absorption silencer enclosed in the gear box.

Loading capacity is commonly given as theoretical capacity, obtained by multiplying the loading cycle with the bucket capacity. Such ideal conditions may conceivably be approached when loading sand or gravel. The capacity of LM56 is a realistic average obtained from numerous time studies of ore loading under normal working conditions.

Smooth working air controls, of a completely new design, are also fitted with special silencers.

Safer to work with

LM56 is carefully designed for maximum safety. One quick movement of a valve lever and the machine is immediately shut off and cleared of air. This makes service and repair work safer and easier. The air control levers are easily locked in a neutral position so that the machine cannot be set in motion by accident. The operator's feet are protected by a safety rail in front of the wheels.

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is viscous and the velocity distribution curve is parabolic. The curve relating $\frac{V_m}{V_c}$ with Reynolds number tends to become asymptotic at a value of $\frac{V_m}{V_c} = 0{\cdot}82$. Under normal practical conditions, the value of Reynolds number is large and therefore a value of $\frac{V_m}{V_c}$ of about 0·80 may be assumed, providing there are steady flow conditions. It is, however, customary in efficiency tests by Pitot tube traverse measurements to determine the pipe factor experimentally by means of exploratory traverses.

The Pitot tube is a comparatively easy instrument to apply on pressure pipes because, by using some form of gland assembly incorporating a shut-off valve, it can be inserted without disrupting normal operation of a power station; any loss of head caused by the obstruction in the flow is, of course, negligible in large pipes although it may have some effect in small, laboratory-type installations. Unfortunately, the instrument is peculiarly sensitive to adverse hydraulic conditions, and a traverse is generally essential for the accurate measurements required in turbine efficiency tests. The flow in pressure pipes is rarely free from foreign matter and the orifices may become blocked, thus causing a delay while the tube is withdrawn and cleared. A further point that should be considered is that for the Pitot tube, the differential head obtained is absolutely dependent on the velocity of the flow, and cannot be altered.

The Pitot tube has, through widespread application, attained a high degree of perfection. As in the case of another point-velocity technique of flow measurement, the current meter method, however, its successful application depends to a very large degree on experience.

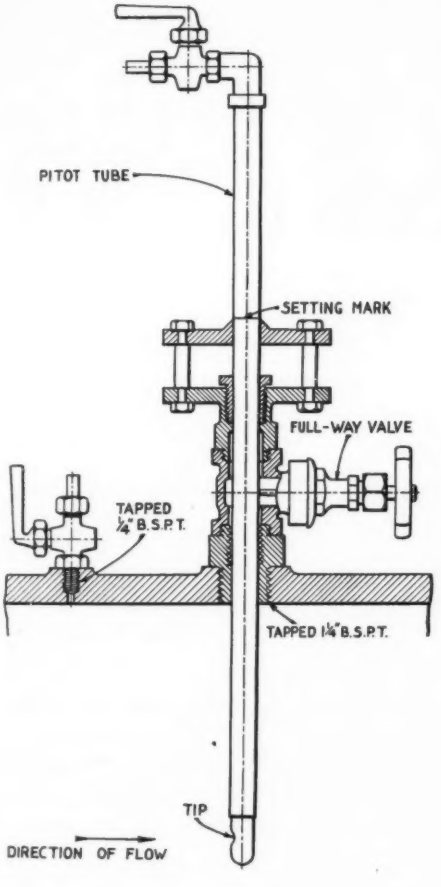

Fig. 8. A Pitot tube tapping

Ultrasonic

All the methods previously mentioned in this article have been used on many occasions for turbine acceptance tests, or for comparative measurements at hydroelectric power stations. There is, however, one method which is still in an early stage of development but which may well eventually surpass all others in simplicity, economy and ease, if perhaps not scope, of application. The ultrasonic flowmeter contrasts the transit times of ultrasonic waves travelling with and against the flow by comparing the phase of two sets of waves. The difference in phase angle is directly proportional to the velocity of flow of the fluid and it is also proportional to the frequency of the ultrasonic waves and the distance.

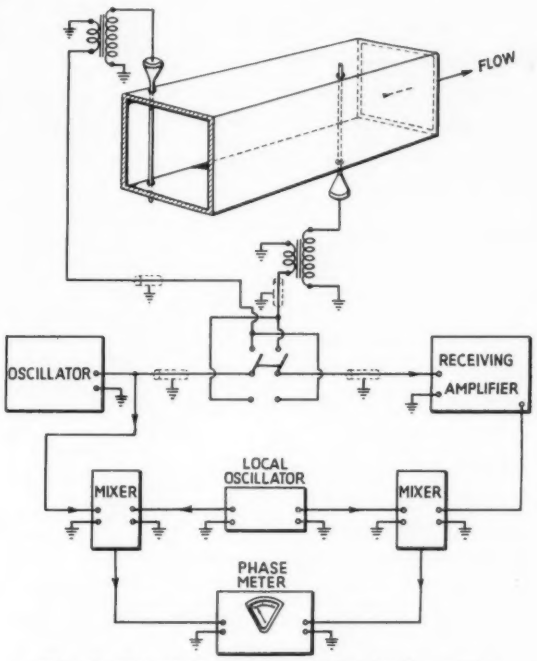

Fig. 9. Principle of the ultrasonic flowmeter

Fig. 9 shows the principle of the method. Two transducer rods are placed in the conduit in which measurement of water velocity is to be made. A transducer, when acting as the transmitter, converts high-frequency alternating current into mechanical vibrations of the same frequency. Conversely, mechanical vibrations picked up by a transducer are converted into alternating current. It is found that the magnitude

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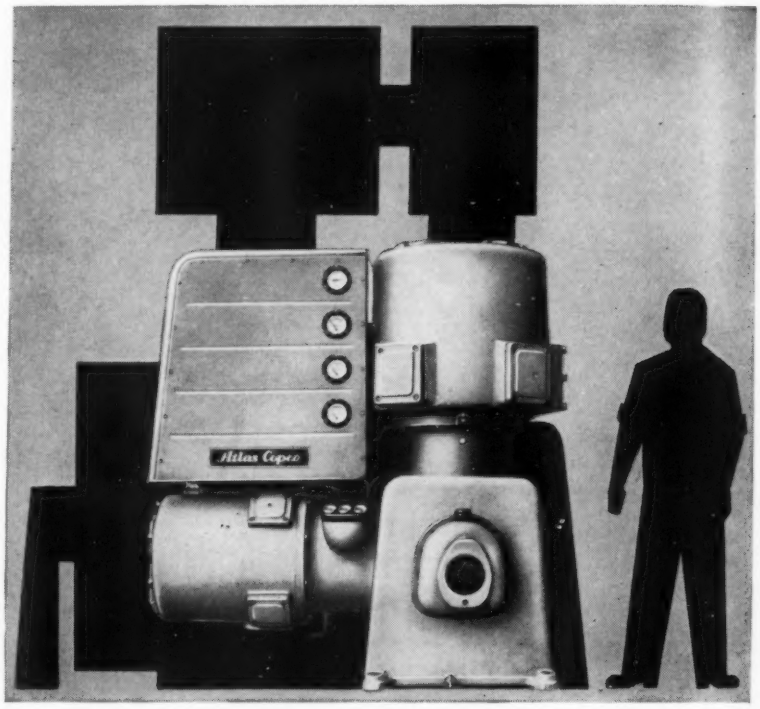

PRINCIPAL DATA

Maximum pressure in psi	Speed in rpm	Low-pressure piston displacement in cfm	Free air delivered at 100 psi in cfm	Cooling water required at 60°F approx in galls/hour	Weight in pounds
125	485	1,300	1,075	440	6,600

Economy of operation

The specific power consumption for the ER6 is 10-15% lower than for most machines of this capacity range on the market. At 4,000 hours running time per year this means an *annual power saving of up to* £360.

In addition the cooling water consumption of the ER6 is very low because of the special design of the intercooler, amounting to only 440 gallons per hour at 60°F. This is less than half the quantity needed for the majority of other similar air compressors.

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The ER6 requires an exceptionally small amount of floor space for its capacity. This reduces the cost of foundation and erection, especially with the use of a flange-mounted motor.

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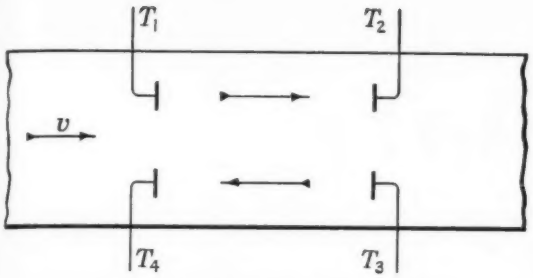

Fig. 10. A differential arrangement

of the phase angle between the transmitted and received signal is a measure of the average velocity of water passing through the conduit, but in practice, to avoid the necessity of measuring the phase angle with the water stationary, two phase-angle measurements are made in quick succession: first, with the downstream transducer receiving the signal of the upstream transducer, and second, with the functions of the two units interchanged.

Part of the signal from the oscillator passes through one channel of the amplifier and mixer to the phase-angle meter. The signal from the receiving transducer passes to the phase-angle meter through the second channel of the amplifier and mixer. The beat method of reducing frequency maintains the phase relationship of the two signals.

If c is the velocity of sound in the still water and v is the velocity of flow between two transducers, then the time of transit is given by:

$$t=\frac{d}{c+v}=\frac{d}{c}\cdot\frac{1}{1+\frac{v}{c}}$$

There are several refinements of this basic principle which are more likely to satisfy the requirements of the high-velocity flows encountered on hydro-electric turbine sets. In the differential arrangement shown in Fig. 10, the time of transit from T_1 to T_2 is $t_1=\frac{d}{c+v}$ while the time of transit between T_3+T_4 is $t_2=\frac{d}{c-v}$. The time difference $\Delta t=t_2-t_1=\frac{2dv}{c^2-v^2}=\frac{2dv}{c^2}$.

In this case, there are no gross errors due to variations in c, but Δt is still affected by these variations, the fractional error in flow measurement (caused by a variation in c of Δc) being approximately $\frac{2\Delta c}{c}$ where $\frac{\Delta c}{c}$ is small.

As mentioned above, measurement of the time difference can be made by the beat method, but a continuous wave system can also be used. In this system, the transmitting transducers are fed from a common source and the phases of the signals arriving at the two receiving transducers are compared. In this case $\Delta\phi\doteq\frac{2\omega dv}{c^2}$ where ω is the angular frequency of the transmitted signal.

In another variant, two pairs of transducers are also used, each pair being formed into an oscillating loop. A short pulse is emitted from transmitting transducer T_1 (see Fig. 11) and is received by receiver T_2 after a time interval t_1. The arrival of the pulse T_2 immediately triggers a further pulse at T_1. The time interval between pulses is therefore $t_1=\frac{d}{c+v}$. The pulse repetition frequency is $f_1=\frac{1}{t_1}=\frac{c+v}{d}$. Likewise, the repetition frequency of the other loop is: $f_2=\frac{1}{t_2}=\frac{c-v}{d}$. The frequency difference between the two loops is easily extracted and is: $\Delta f=f_1f_2=\frac{2v}{d}$. This result is independent of the velocity of sound, thus greatly simplifying the whole procedure.

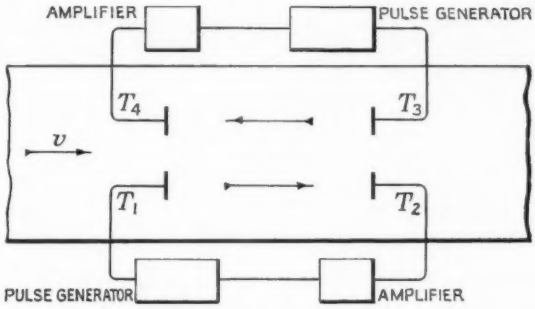

Fig. 11. Meter with oscillating loop

The concept of the ultrasonic method of flow measurement represents a marriage between the sciences of electronics and hydraulics. At the present stage of development, there are many problems to be overcome if high-velocity flows in large conduits are to be successfully and accurately measured by this method. It is not the purpose of this journal to discuss the more detailed functions of an electronic apparatus, but it is interesting to note the possible future advantages of the method insofar as turbine-efficiency tests are concerned. For instance, it might well be feasible to measure high-velocity flows instantaneously without tapping a pipe, and without in any way disrupting the normal operating schedule of a power station. The cost would be comparatively low and measurements would be permanently available to assist in maintenance and calculations of losses due to cavitation or other causes. On the other hand, at the present stage of development, the ultrasonic flowmeter may not be accurate for fluctuating velocities because the ultrasonic beam takes a linear path across the conduit and its deflection or time delay is integrated linearly across the profile. The volume flow, however, is dependent on the two-dimensional profile; that is, it is an area effect. The flowmeter does not, therefore, take the velocity profile into account and prior calibration would therefore be necessary.

Conclusion

Galileo is reported to have said "I have met with fewer difficulties in observations relating to the movements of heavenly bodies, notwithstanding their stupendous distances away, than in investigating the

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movement of flowing water which takes place before our very eyes." Such a comment would be endorsed by any hydro-electric engineer conversant with methods of measuring turbine efficiency.

This article has mentioned only some of the methods that can be used, but the diverse techniques and principles involved, even in this selection, show how much still has to be done before a simple, accurate and inexpensive method can be applied universally.

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Book Reviews

Hydrology for Engineers. By Ray K. Linsley, tr., Max A. Kohler and Joseph L. H. Paulhus. Published by McGraw-Hill Book Company Inc., New York, Toronto and London, 1958, 321 pp., 177 ff. Price 62s. net.

This book presents the background theory and the specific techniques of engineering hydrology in simple readable terms. It provides a sound theoretical foundation in hydrology and, at the same time, emphasises practical, well-tested methods for the application of hydrology in engineering. Special emphasis is given to techniques which utilise correlation methods and eliminate judgment to the maximum possible extent.

An interesting chapter on stream flow discusses stage gauges and current meters, and the problems arising in using current meters for accurate measurement. When ice covers a stream, a new friction surface is formed and the stream becomes a closed conduit with lower discharge because of the increased hydraulic radius. The underside of the ice sheet may be extremely rough if ice cakes are tilted and then frozen together at random. Movement of the water under the ice gradually erodes the surface until the latter is smooth. If the stage falls, leaving the ice as a bridge across the stream, the stage-discharge characteristics return to those of a free stream.

Certain sections of this book contain information of great value to the hydro-electric engineer. For instance, the chapter on precipitation gives a clear explanation of this natural phenomenon. Although water vapour in the atmosphere is a necessary factor in the formation of precipitation, it is by no means the only requirement. Moisture is always present in the atmosphere, even on cloudless days, but rainfall is only induced when some mechanism cools the air sufficiently to cause condensation and droplet growth. Condensation nuclei are also necessary, but they are usually present in the atmosphere in adequate quantities.

The authors have explained the hydrological criteria for the designs of storage reservoirs and spillways, and there are also interesting sections on reservoir sedimentation, river forecasting, components of run off, and on many other aspects of the science of hydrology.

The book follows the pattern used by the authors in "Applied Hydrology" (McGraw-Hill, 1949), but the scope is limited to those aspects of hydrology which specifically concern the engineer. Discussions have been simplified for use at an elementary level, but generous references are given for further study.

Chambres d'Equilibre (Surge Tanks). 2nd edition. By A. Stucky. Published by Sciences & Technique, Rue du Petit-Chêne 11, Lausanne, Switzerland. 182 pp., 88 ff. 1958. Price, paper covers, Sw.F. 32; bound, Sw.F. 38.

In 1951 Prof. A. Stucky, of Lausanne Ecole Polytechnique, published privately his lectures on surge tanks. The first commercial edition appeared in 1952 and the second edition of this work is now being published.

These lectures cover the well-known basic equations for surge oscillations and the general treatment closely follows the classical treatise by Calame and Gaden (1926) but many additional examples are solved with the Schoklitsch graphical method. Developing the theory of surge stability (Thoma's formula), the author again follows Calame and Gaden's earlier publications, checking results against the graphical method. Additional use is also made of Gardel's doctor thesis, published in Lausanne in 1956, in the period between the first and second editions of Prof. Stucky's lectures.

The basic theory is extended to surge tanks with expansion chambers, to differential surge tanks and to restricted-orifice tanks. A valuable addition to the second edition is chapter II, dealing more extensively with the water-hammer theory (somewhat neglected in the 1951/52 editions) and with the Schnyder-Bergeron graphical method.

The final chapter, which is also new, includes three interesting examples of surges in more intricate hy-

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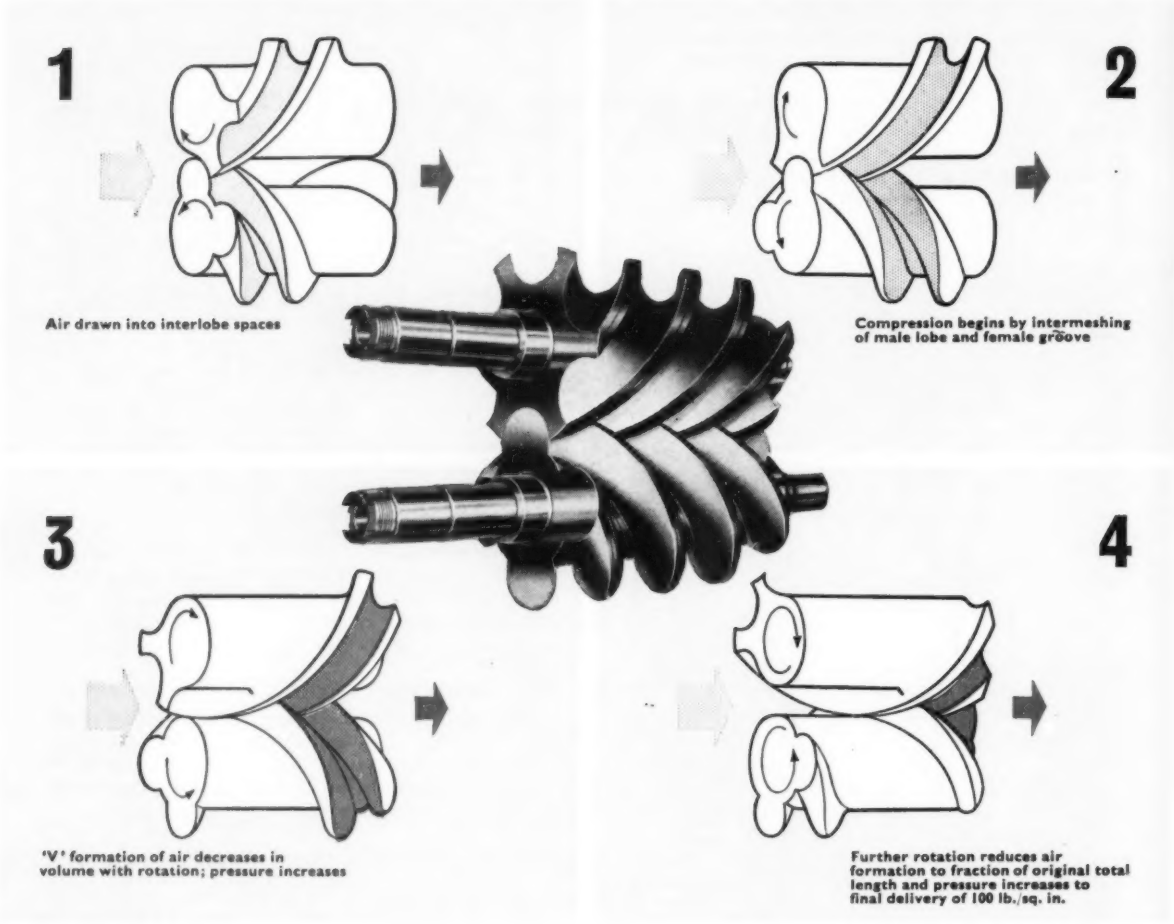
1
Air drawn into interlobe spaces
2
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3
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draulic systems. These examples are, in fact, practical problems which have arisen during the design of the Grande Dixence power system.

The book gives good coverage of the usual basic syllabus on surges and water hammer. The many examples and practical problems are presented with skill and the work can be recommended to students and practising engineers alike.

Hydrogéologie. Second edition. By P. Fourmarier. Published by Masson et Cie, 120 Boulevard St.-Germain, Paris, and H. Vaillant-Carmanne, S.A., 4 Place St.-Michel, Liége, 294 pp., 164 ff. Price 3,000 francs (paper covers).

This book was first published in 1939 and the new edition is a revised and up-to-date version of the original. Hydrology as a science is considered mainly from a scientific and practical viewpoint and advanced theoretical considerations are kept to a minimum.

The first edition included a chapter devoted to the artificial treatment of water and there has been very considerable progress in this field during the past 20 years or so. This section of the original book has now been completely revised by Professor Edmond Leclerc and it is now included as an appendix.

Although it does not deal with a subject of direct interest to hydro-electric engineers, this book contains much valuable information for those concerned with civil engineering work, and particularly for engineers interested in tunnels and underground power stations where too much water, or too little, are often problems of paramount importance.

Abnahmeversuche an Wasserturbinen—VDI–Wasserturbinenregeln (Water-Turbine Acceptance Tests—VDI's Water-Turbine Regulations). Published by DNA (German Standards Committee), Berlin, W.15; 8 in. by 11·5 in., 30 pp., 44 ff. Sole distributors: Beutch-Vertrieb G.m.b.H., Berlin, W.15, and Cologne. Price in Germany DM. 13·20.

This is a revised edition of the Standard Regulations first published in 1930, to which the VDI Flow Measurement Regulations have been appended. The revision committee set up by VDI under the chairmanship of Dipl. Ing. E. Treiber included manufacturers' and users' representatives, as well as other specialists and delegates from Austria, Sweden and Switzerland. A short introduction dealing with generalities is followed by 5 sections of which the first, Definitions and Nomenclature, also gives the corresponding symbols and units. Otherwise the range of rules is the same as specified in the I.E.C. Hydraulic Turbine Test Code, i.e., Nature and Extent of Guarantees, Conditions to be fulfilled in Acceptance Tests, Procedure applying to Model Tests, and Methods of Measurement. Decimal classification is used throughout the brochure and by a glance at the contents the reader is enabled to find at a glance any particular section or subsection of the rules he wants to refer to. Print and illustrations leave nothing to be desired.

Hydraulics Research Papers Nos. 1 and 2. Published by the Department of Scientific and Industrial Research, Charles House, 5-11 Regent Street, London, S.W.1. Paper No. 1, "Resistance of Fluids Flowing in Channels and Pipes," price 8s. 6d. (by post 9s.), U.S.A. $1·53. Paper No. 2, "Charts for the Hydraulic Design of Channels and Pipes," price 12s. (by post 12s. 8d.), U.S.A. $2·16.

The Hydraulics Research Station is introducing a new series of Research Papers, the first two of which deal with the flow of water in open channels and pipes. Design engineers will find them both valuable for reference purposes. Although many formulae are now available they are generally of limited application and designers have experienced difficulty in selecting an appropriate one and much confusion has been caused. In the 1930s, Dr. Colebrook and Professor White working at Imperial College, London, employed the then new theories of turbulence to bring almost all the problems of hydraulic friction within the scope of one equation. This equation is recognised as the best so far devised but unfortunately it has proved to be rather too complex to be popular. Attempts have been made to express it in an easily useable form, but none was completely successful.

The Hydraulics Research Station has now solved the difficulty by developing the formula so that it can be expressed in graphical form. Paper No. 1 explains this development after fully reviewing and discussing the subject of hydraulic friction. Paper No. 2 contains 28 design charts, based on this development of the Colebrook–White formula, covering a wide range of conditions, materials of construction and cross-sectional shapes for conduits ranging from 80 ft. diameter hydro-power tunnels down to 1 in. diameter copper piping. Many examples of hydraulic designs are given in this publication, including concrete-lined channels of rectangular and trapezoidal cross sections and storm drains flowing with a free-water surface.

Pumps, Fans and Compressors by A. de Kováts and G. Desmur, translated from the original French by R. S. Eaton. Published by Blackie and Son Limited, 17 Stanhope street, Glasgow, C.4. 327 pp. Price 45s. net.

This book discusses the principles, design and construction of axial flow and centrifugal pumps. Chapter 27 deals with pumps operating as turbines or reversed pumps and Chapter 21 gives a brief description of hydraulic storage units. The book is profusely illustrated and it will be of considerable interest to all those concerned with pumps and their applications.

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Selecting Demolition and Construction Equipment

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The main characteristics of this plant are: Equipped discharge 440 cu. m. per sec.; gross head 10·2 m.; normal storage level 161·00 m. above sea level; length of backwater 12 km.; installed capacity 39,750 kW; yearly production 225 million kWh. The layout of the works from right to left bank is: a three-span spillway block, the power house proper and its erection pit, an outbuilding built on to the downstream wall of the power house, and the transformer station. The plant is of the low-deck type, power house and spillway block being served by the same 100 ton gantry crane. The spillway block has 18 m. openings separated by 6 m. piers and controlled by sector gates which are fitted at their top with a folding shutter acting as overflow. The roof of the power house is a ferro-concrete slab supported by a framework of steel beams. The openings giving access for the gantry crane to the erection pit and to each of the generating units are closed by hoods moving sideways on rollers. Since the works extend to most of the width of the river bed, the power canal is practically reduced to an inlet sill placed at 151·00 m. directly upstream of the wing wall, which it connects to the left-bank wall. Each of the three vertical-shaft turbo-alternators consists of a three-phase 14,500 kVA alternator operating at 5·7 kV, and of a 13,250 kW Kaplan-type turbine. The erection pit is equipped with a 5 ton travelling crane. The drainage pumps, workshop, offices, first-aid post, staff room, and sanitary installations are housed in the outbuilding. An access road laid on the left bank of the river leads to a platform at the level of the roof of the erection pit, which also carries the transformer bench. Adequate dwelling accommodation for the operating personnel has been built on the right bank. The installations of the construction plant set up on the left bank with all the necessary workshops, offices, concrete laboratory, etc., are described in the article, together with the water, electricity and compressed-air supply systems. Preliminary work began in April 1955, and construction work followed in three phases: 1. First two spans of the spillway block and upstream bank wall, the construction pit being protected by a double curtain of Larsen-IV steel piles. Concrete work began at the end of November 1955, and in spite of a total interruption during the very cold period of February 1956, pulling up of the protection curtain was completed on July 17, 1956, except for the sections comprised in the curtain of the second phase; 2. The river flowing through the two first spans of the spillway and the space between the pier nearest to the power-house block and the left bank, work started on the third spillway span and the wing wall, and initial excavating and concreting for the power house proceeded under the protection of a curtain of the same type as for the first phase; 3. Concreting work for the power station proper and the outbuilding under the protection of a curtain connecting the wing wall and the left bank. Pulling up of the piles was completed upstream on May 12, 1958, and downstream on July 15. The first generating set was put on the line on August 10, the second and third being scheduled for commissioning two months and four months later respectively. The article concludes with a paragraph on the construction of the bank wall on the right bank downstream, and a note on the addition of fly ash to mass concrete. The Kaplan-type turbines were supplied by Société Neyrpic, and the alternators and transformers by the Alsthom concern. (H. Duserre, A. Ducret and F. Portier, *Construction*, Vol. XIII, No. 11, November 1958, p. 321, 8 pp., 13 ff.)

Water Power in U.S.S.R.

Following a short introduction, the author describes the distribution of water-power resources in the Soviet Union, of which no less than 85% is available in Siberia through such great rivers as Ob, Yenissei and Lena, and their tributaries. This figure corresponds to a yearly production of 1,180 billion kWh out of an all-Russian aggregate of 1,721 billion kWh. The Volga, Dnieper, Angara-Yenissei and Ob-Irtish developments are described in separate paragraphs. The chapter on general considerations includes sections on planning, construction, concrete technique, winter concreting, construction plants, cofferdams, weirs in run-of-river plants, and high-head plants. From the note concluding the article, we learn that new Russian installations in 1956 totalled 5·5 million kW, the yearly increase of installed capacity being assessed at 7 to 8 million kW for the following years, to reach 10 million kW in 1963. Production is expected to reach 500,000 million kWh in 1965. From the speech of the Soviet Prime Minister at the inauguration of the Kuibyshev plant, it would appear that the general economy of the country is beginning to feel the adverse effect of this hasty development of water-power resources. Mr. Khrushchev announced restrictions in the building of major hydro-electric developments and the "mothballing" of some plants now in course of construction. Stalingrad, Bratsk, Kremenchug, Wotkink and Bukhtarma, totalling 7 million kW, are to be put into operation within the coming seven years. Krasnoyarsk is to be completed during the second Seven-Year Plan. Work at Saratov, on the Volga, and Dnieproserzhinsk, on the Dnieper, appears to have been suspended. (Dipl. Ing. H. Grossen, *Schweizerische Bauzeitung*, Vol. 77, No. 51, December 20, 1958, p. 769, 5 pp.; No. 52, December 27, 1958, p. 785, 3 pp.; 11 ff.)

Rockfill v. Concrete in Dam Building

In the first section of this article, the author discusses the field of application of rockfill valley-closing dams, and reviews the criticisms made from a general viewpoint on this class of structure. His study is guided by the astonishing fact that in Europe relatively few earthfill and still fewer true rockfill structures have been constructed for impounding purposes. For instance, out of 200 Alpine dams set up in connection with French, Italian, Swiss and Austro-Bavarian hydro-electric schemes, there are no more than 27 "fill" structures, including very few true rockfill dams. As such are considered to be dams in which

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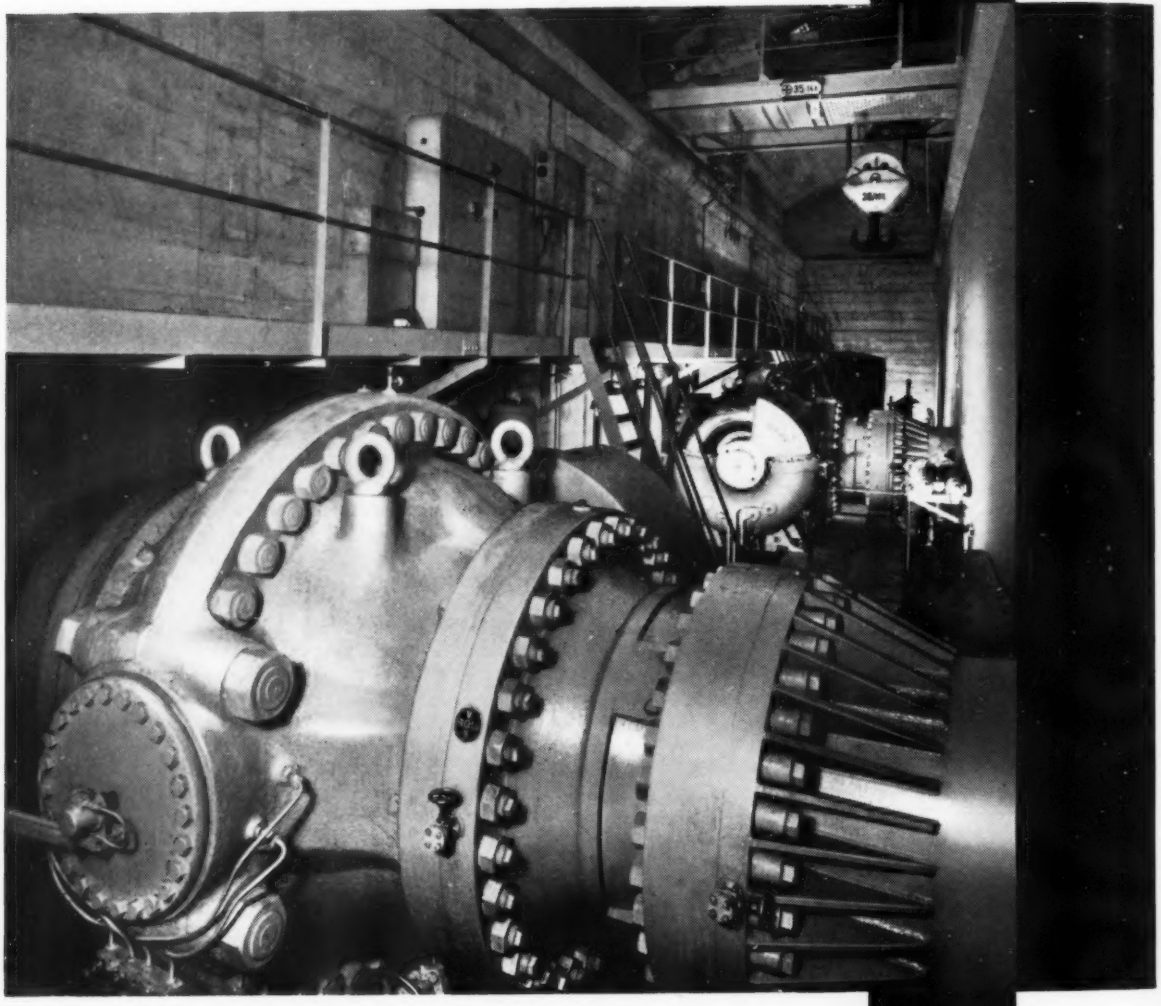

Valve chamber with rotary slide sphériques valves as turbine shut-off device in a swiss power station

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the fill consists of homogeneous fragmentary rock 10 to 60 cm. in size without any binding material, sand or round gravel, or any combination of dumped material varying in friction coefficient. The prejudice against rockfill dams arose out of the bursting of older fill structures, but has today no justification owing to stricter building regulations, particularly with regard to foundations, the advance of impervious facings or cores, and the practical experience collected in the building of high dams. This view is supported in Germany by such acknowledged experts as Professors Keil, Press, Tölke and Dipl. Ing. Lewin. The advantages and drawbacks of impervious cores and upstream facings respectively are discussed. In section II. the design and construction of rockfill dams with filling up of the voids is reviewed; such a structure with faces at a natural slope and fill consisting of coarse, irregular shaped rock, without any round material, is safe from settling, and imperviousness is ensured by a slanting zone inside the upstream slope of the dam, in which the voids are filled with impervious, non-erosive and strongly adhesive material. (Dr. Ing. Erich von Posch, *Die Wasserwirtschaft*, Vol. 48, No. 13, October 1958, p. 339, 5 pp., 2 ff.)

Bia River Development

This French Ivory Coast development is to operate in conjunction with existing thermal plants, and is subsequently to ensure the supply of the town and region of Abidjan in electric energy. It will consist at its ultimate stage of the Ayamé or upstream works described in this article, and of a downstream plant to be set up at a later date. The Ayamé power station, built at the toe of the dam, has an installed capacity of 19,200 kW designed to produce yearly 80 million kWh from a nominal head of 19 m. (minimum 14 m., maximum 22 m.); this is a relatively small plant, but its civil-engineering works present highly interesting features. The storage reservoir, which will cover an area of 9,000 hectares at 90·5 m. above sea level, is created by a concrete buttress dam extending from the left bank of the river 310 m. into the bed, the closure of the valley being completed by a compacted-earth dike. The dam consists of 28 monolithic self-supporting blocks 10 m. wide, with buttresses broadening on the upstream side. Each block connects with its neighbour by a flat joint 1·5 m. wide closed upstream by a rubber gasket. On the left bank, as the inadequate rock surface ruled out a gravity abutment, the closure of the storage has been effected by a screen poured into a concrete-lined excavation, thus forming an extension of the dam, and penetrating the strata which overlie the rock *in situ*. On the right bank there is no rock surface, and the concrete dam backs the compacted-earthfill dike which closes the reservoir above a wide lateritic spur narrowing the valley just opposite the dam site. The fill material obtained from deposits near by is laterite, which, in addition to very favourable mechanical properties, has the advantage over European clays of not swelling in the presence of water, thus maintaining a strong resistance to shearing. The distribution of this laterite fill in the body of the dike is such that the central core is made of fine clay and covered by clay containing 25% of nodules. The faces are protected by a rubble-stone apron 3 m. thick upstream and 1 m. downstream. Seepage water is collected by a filter and deep drain. The joint between dam and dike is achieved by means of a concrete reinforcement anchored in No. 1 block of the dam. The two-phase procedure adopted for construction work in the river bed was determined by extensive model tests carried out at the Chatou National Laboratory of Hydraulics; it is described at length at the end of the article. Work on the Ayamé plant began in September 1956 and the article deals with the progress of work as achieved by June 20, 1958. The closure of the valley on the right bank seems to have been completed; the closure on the left bank is to be effected in the second phase of construction work which does not appear to have begun. (J. Vassivière and G. Feillée, EDF, *Construction*, Vol. 13, No. 11, November 1958, p. 329, 4 pp., 8 ff.)

Lester A. Pelton

Lester A. Pelton, the original designer of the well-known turbine that bears his name, was born in Ohio in 1829. At the age of 20, he became a gold prospector and lived in the small town of Camptonville, California; he was only moderately successful in his search for gold and so, being interested in mechanics, he turned his attention to the construction of water-driven stamp mills. He progressed rapidly and soon saw that the relatively common hurdy-gurdy runner of that time left a great deal to be desired in efficiency. During the winter of 1877 and 1878, he constructed over 40 different models of runners and buckets, and eventually evolved a bucket made of two symmetrical sections which could be driven efficiently by a tangentially-opposed jet. Pelton patented his invention in 1880 and soon afterwards the first Pelton turbines were manufactured by the Nevada City Company of G. G. Allen and Co. In 1887, Pelton left Nevada City and founded a new firm in San Francisco which subsequently became known as the Pelton Water Wheel Company. Pelton died in March 1908 at San Francisco, at the age of 77. (Prof. L. A. Haimerl, *Energie*, No. 12, December 1958, p. 3, 5 pp., 13 ff.)

World's Largest Single-Phase Generators

These are the two 50,000 kVA, 16 c.p.s. generators supplied by the Oerlikon Engineering Company to the order of the Swiss State Railways for the underground power station of the Göschenenalp-Göschenen stage of Göschenen Kraftwerk AG. (WATER POWER, February 1955, p. 79, and January 1956, p. 39). Their four-pole rotors, 3,030 mm. in diameter, weigh, complete with poles, about 160 tons each. Because of their size, the rotors will be transported by rail in halves and assembled and wound on site; rotor winding is of the single-layer grid type with one wire per groove. Groove insulation material, as in all Oerlikon machines, is "Orlitsa," a compound of "Samika" and synthetic resin. The main advantage of this compound over the usual mica insulators with thermoplastic binding material lies in its high thermomechanical strength. The generating equipment of the plant is completed by two 53,300 kVA three-phase generators operating from the same stage at 0·75 p.f. 10 kV and 50 c.p.s. and at the same speed as the single-phase units, 500 r.p.m., and by two smaller generators operating from the Andermatt-Göschenen stage, a 20,000 kVA single-phase generator feeding the railway system, and a 22,000 kVA three-phase generator

feeding the overhead line which connects the Wassen power station with Rathausen. All six generators are of the vertical-shaft sunk type and each is driven by a three-nozzle Pelton wheel. An interesting feature shared by all the generators of the Goeschenenalp-Göschenen stage is that their lost heat can be collected by a heat pump to heat the bottom floor of the power station and the control room. (H. Borel, *Technische Rundschau*, Vol. 51, No. 2, January 16, 1959, p. 33, No. 2.)

Analysing Arched Dams

In 1938, the U.S. Bureau of Reclamation published in book form its "Trial Load Method of Analysing Arched Dams," in which the investigation of radial, tangential and angular loads is carried out by adjusting the deformation of the components of the dam by means of successive approximations, these components being considered first as arches and then as cantilevers. Proceeding from this method and with a view to reducing the extent of the trial and error procedure and eliminating certain practical difficulties, the author of this article advocates a method in which these loads are calculated by means of linear equations enabling the deformations at the points of intersection of the arches and cantilevers to be determined directly. By applying this method, it is claimed that a constant check can be kept on the results obtained in the course of the analysis, and the necessary adjustments can be made in the instance of loads differing from those assumed at the outset. Ultimately, the calculated results can be readily compared with those obtained by direct measurements. The main advantage claimed for this method is that it is much quicker than the trial-load procedure. The time taken by the analysis can even be further reduced by dividing the operation between several computers. (V. Mladyenovitch, *Travaux*, Vol. 42, No. 290, December 1958, p. 1037, 7 pp., 7 ff., 4 tables.)

Facing the Montélimar Canal

The power canal of the Montélimar plant branches off from the left bank of the Rhône near Ancone and runs roughly parallel with the river to feed the Chateauneuf-du-Rhône power station, whence a 1,800 m. tailrace returns the water to the Rhône. The bed of the canal is sufficiently impervious, and facing has been limited to the inner slopes of the embankment in order to protect them from wave action. The embankment consists of dikes made either of gravel, silt, or of a combination of both materials. Higher than those of Donzère, the dikes reach 16 m. in the vicinity of the power station. They exceed the normal pool level by 1 m., and are topped by a 1 m. spoilbank to prevent waves caused by the mistral or the wash of ships from overflowing. The area of slopes to be faced amounted to 560,000 sq. m. for the power canal proper, 30,000 sq. m. for the tailrace, and 20,000 sq. m. for the spoilbank, in all a total of 610,000 sq. m. From the three types of facing selected after extensive testing for the Donzère-Mondragon canal, material *in situ* mixed with cement, overlapping ferro-concrete slabs held together by a continuous wire grid, and bituminous concrete, the third one only was retained for Montélimar as having given the best results and being the most economical. In fact, its continuity, flexibility and permeability, enable it to follow closely deformations due to settling without cracks or defacements resulting from uplift and extrusion. The preparatory work, coating of the aggregates, and facing proper are described in detail. In spite of the fact that the canal was filled with water shortly after completion of the dikes, the facing behaved satisfactorily and adapted itself very well to normal settling. There were very few instances of subsidence and sliding, but since it was possible to detect their origin at once, they were promptly remedied. (A. Hennocque, *Construction*, Vol. XIII, No. 5, May 1958, p. 139, 6 pp., 10 ff.)

Applying Electronic Computers to Surge Studies

In this short note, the author discusses the possibility of using electronic computers in the investigation of surges in supply systems comprising a surge tank, and gives as an example the tests made on a model of the supply system of the St. Martin-Vésubie project which includes a surge tank and two shafts acting as additional surge chambers. These investigations were effected at the request of the Alpes-III Section of EDF by the SOGREAH concern, who utilised the IBM electronic computer of their Scientific Division. Their purpose was to analyse the surge regimes to be expected in actual practice, and more particularly, to examine the most unfavourable stages of valve closing and opening. The computer enabled over 20 different instances of valve closing and opening to be analysed within the short time laid down by EDF, and demonstrated the necessity of altering certain features of the project, especially in consideration of the magnitude of the surges ascertained in the shaft sited at a right angle with the Spaillard intake. In his conclusion, the author claims that electronic computers enable otherwise extremely tedious problems to be solved easily and quickly, provided they are under the control of experts specialising in the correct programming of computations, and familiarised with the technique involved in the instance under consideration. Further, this method makes it always possible to record the main or secondary computation programmes, so that when eventually a similar instance presents itself, work is considerably simplified. (M. Paccard, SOGREAH, *La Houille Blanche*, Vol. XIII, No. 6, November 1958, p. 674, 4 pp., 3 ff.)

NOTE.—A fairly comprehensive description of the St. Martin-Vésubie Development appeared in WATER POWER, November 1958, p. 439.

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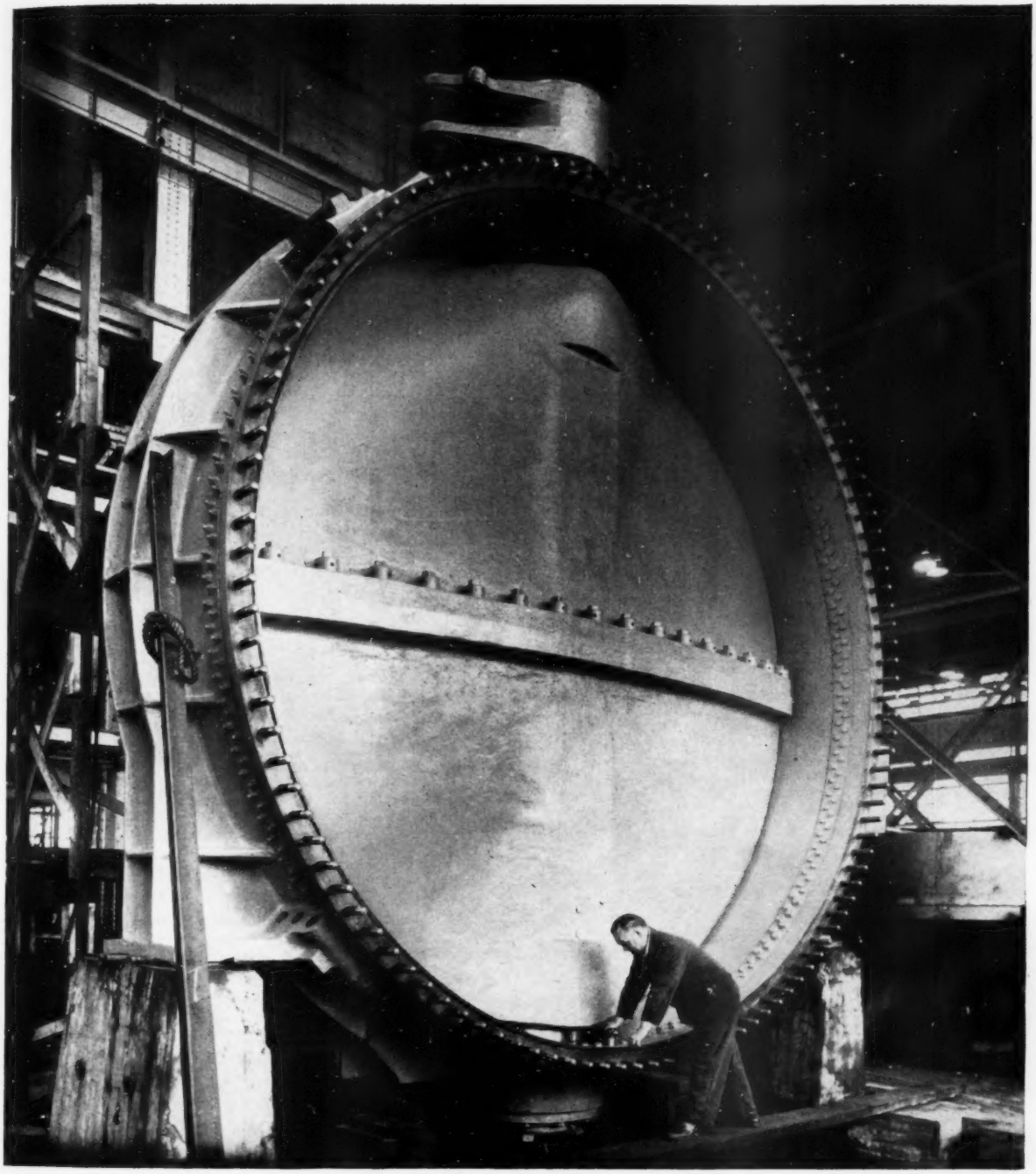

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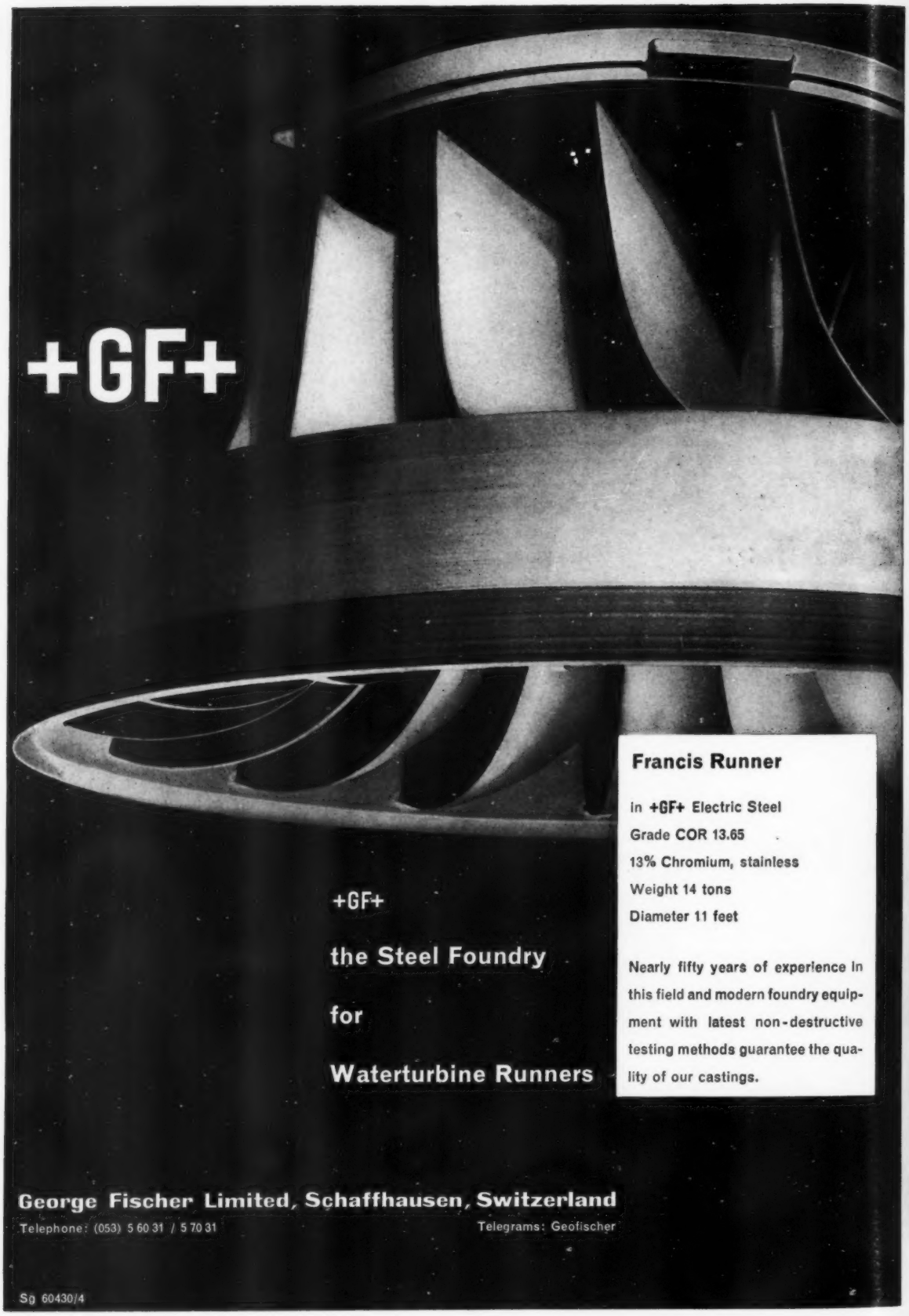
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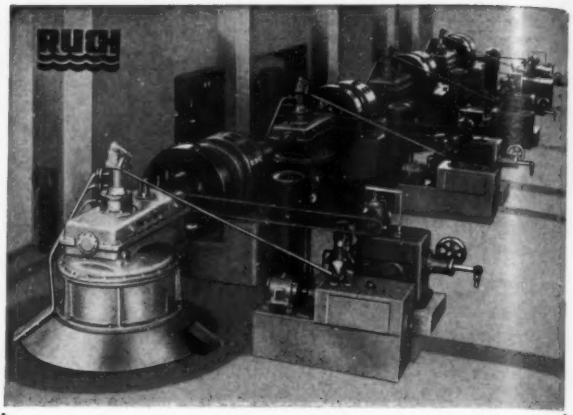

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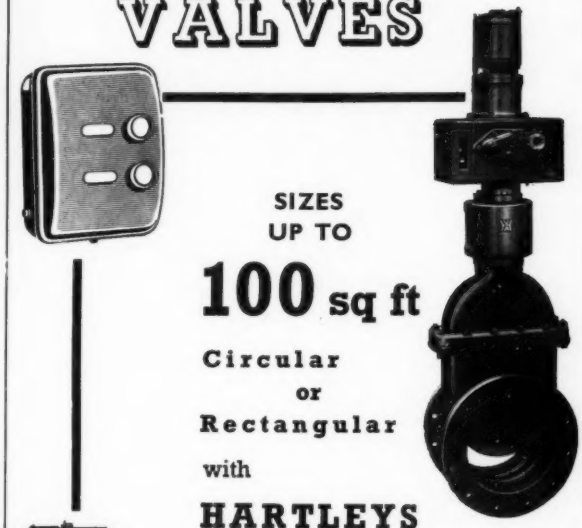

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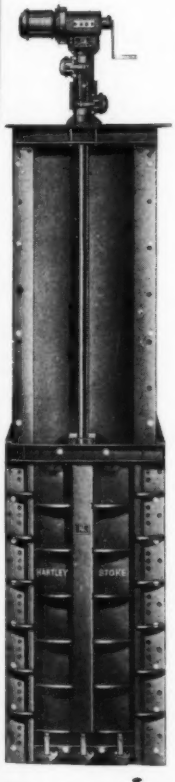

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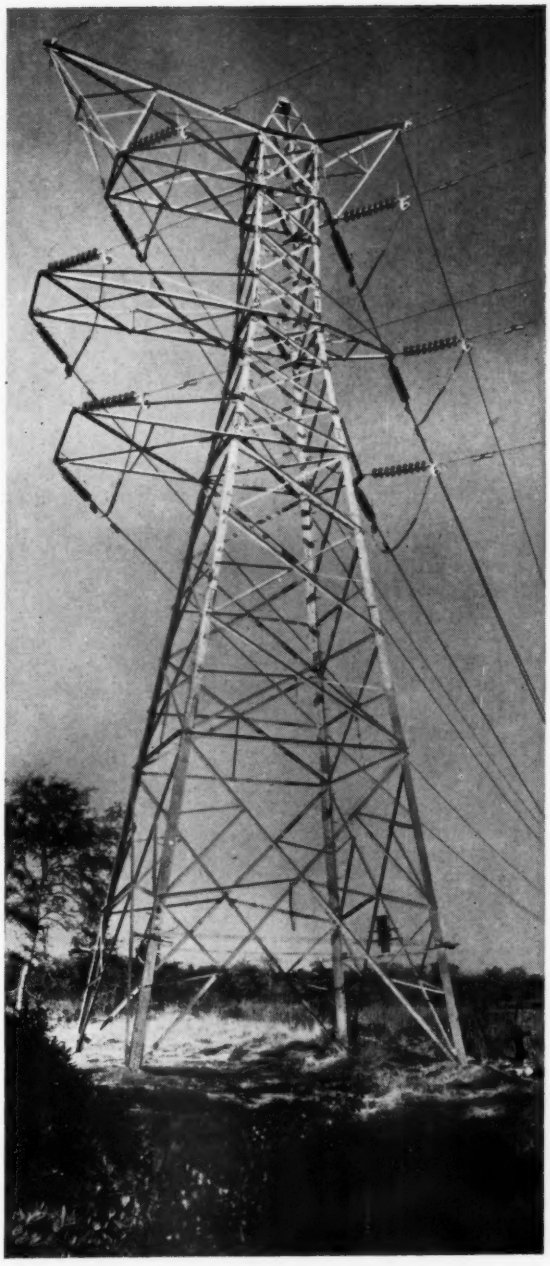

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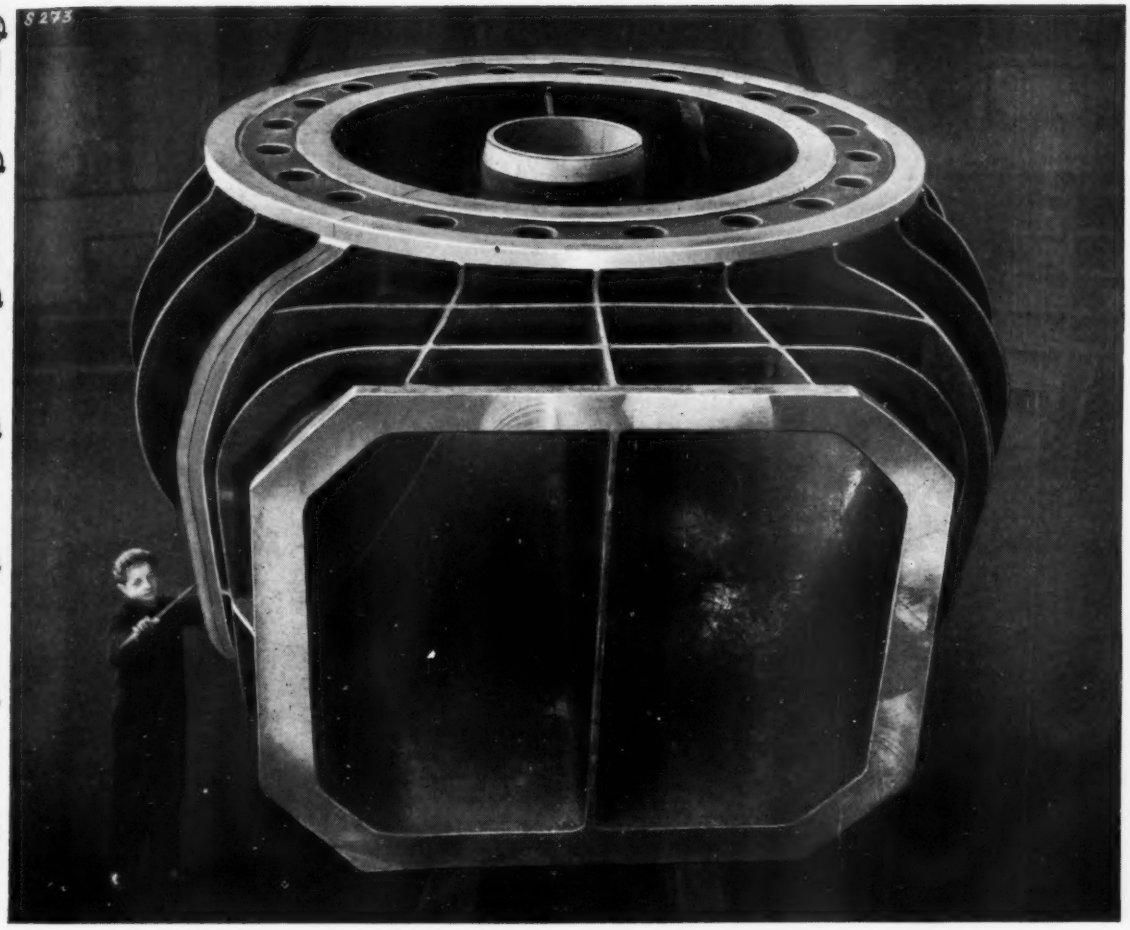
Yorkshire Electric Transformer Co. Ltd. ... ... —

LONDON: Published by TOTHILL PRESS LIMITED, Proprietors of WATER POWER, at 33 Tothill Street, Westminster, S.W.1., and Printed by W. & H. SMITH LTD., Evesham, Worcestershire. *March, 1959*



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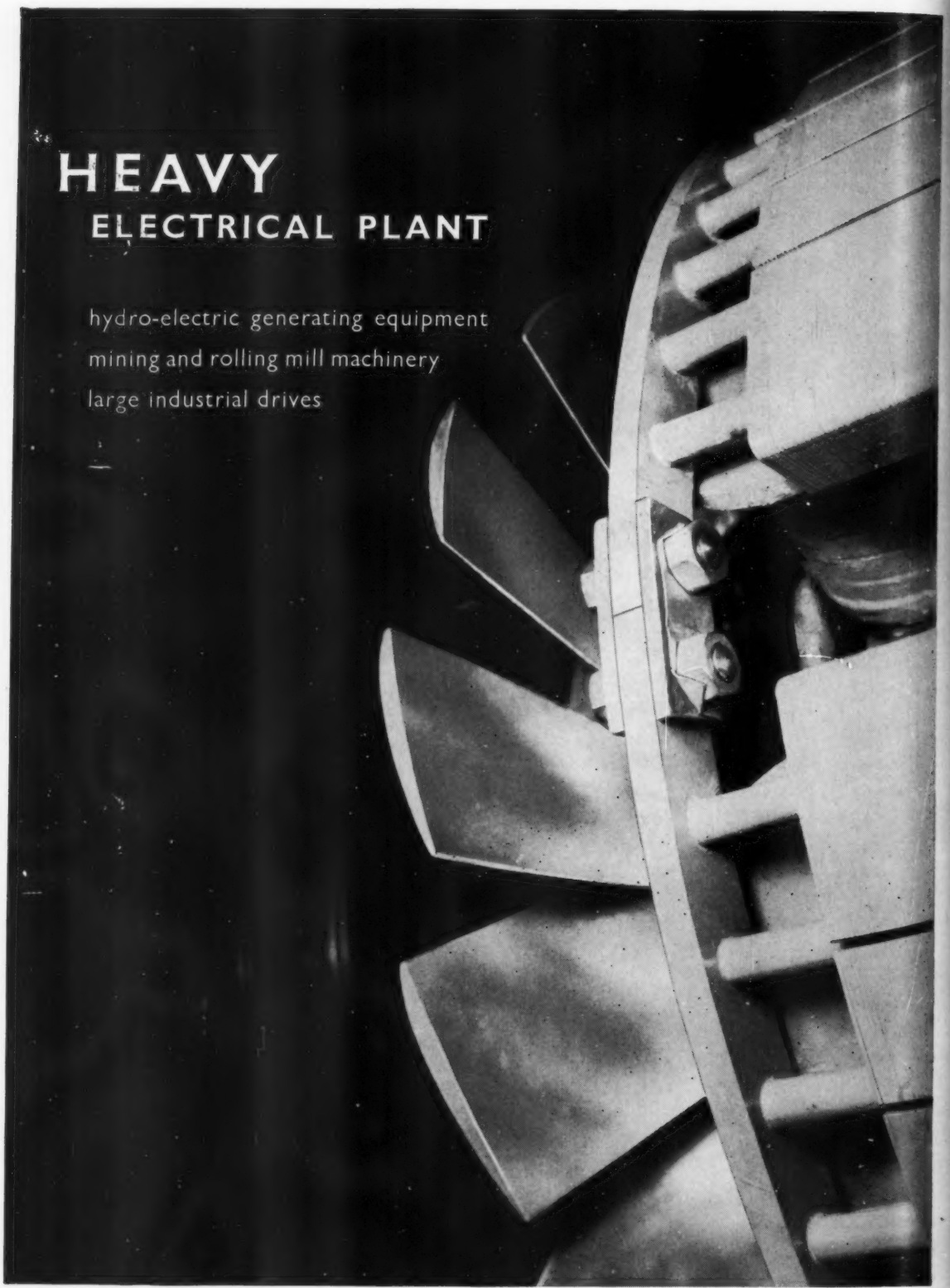

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